HYDROLOGY AND HYDRAULICS REPORT

Little River #3112

over

Little River

Gorham, Maine

WIN 023909.00

Maine Department of Transportation
Bridge Program

HYDROLOGY REPORT

The Little River has a drainage area of approximately 49.8 square miles. The Presumpscot River just downstream as a drainage area of 577 square miles with dams and storage.

Little River peak flows for the Q1.1, Q2, Q5, Q10, Q25, Q50, Q100 and Q500 were provided by MaineDOT. However, during the calibration process for the existing bridge hydraulics model, it was necessary to adjust the design flows upward towards the higher confidence levels to simulate flood elevations at the bridge reported by USGS Water-Resources Investigation Report 97-4189, titled "Flood of October 1996 in Southern Maine" (Report) and confirmed by residents.

The Report also stated that flows released from Sebago Lake were insignificant. Which suggests that the small to intermediate tributaries significantly contributed to the peak flow.

1. Peak Flow during 1996
    i. Based on the Report, the peak flow just downstream of the Saccarappa Dam was approximately 23,300 cfs.
    ii. The upstream boundary conditions had a constant discharge and subcritical regime. (Inlet-Q type condition used).

2. Upstream Boundary Condition – Little River
    i. Based on the Report, there was approximately 17-inches of rain that fell during a 36-hour period (10/20-10/22 of 1996).
    ii. Based on Streamstats, the upper limit confidence interval for the 500-year return is approximately 9,340 cfs. Based on a drainage area of 49 sq. miles, this is deemed reasonable for a rainfall of 15-inches during a 48-hour event.

3. Additional Internal Boundary Conditions
    i. Internal Sink Boundary Conditions were used to add flow to the model domain downstream of the Little River Confluence at the Mosher Brook and Inkhorn Brook confluences assuming a 500-year return per Streamstats.
        - Mosher Brook – 336 cfs
        - Inkhorn Brook – 752 cfs

4. Upstream Boundary Condition – Presumpscot River
    i. Information regarding the Mallison Falls Dam is found on the Low Impact Hydropower's institute website and has a crest length of 280 ft +/-. The Report stated a headwater elevation of approximately 95.5 ft (NGVD29). The Mallison Falls Dam has a crest elevation of approximately 90.5 ft (NGVD29). The following weir equation was used to determine the flow over dam during 1996 assuming an additional 0.39 ft (5fps) of velocity head.
        - $\mathrm{Q} = 3.2\mathrm{LH}^{3/2}$
        - $\mathrm{Q} = 3.2 * 280 * 5.39^{3/2} = 11{,}212 \text{ cfs}$
    ii. Just downstream of the Mallison Falls Dam, the Colley Wright Brook is found and assuming a 500-year return, a flow 1,250 cfs was calculated using Streamstats.
    iii. The Mallison Falls Dam has a maximum turbine capacity of 390 cfs.
    iv. A total flow of 12,852 cfs was calculated for the Upstream Boundary Condition Flow but a flow of 12,872 cfs is used for mass conservation.

SUMMARY

	Little River		Presumpscot River	
Q1.1	1,004	ft^3/s	2,449	ft^3/s
Q2	2,040	ft^3/s	5,310	ft^3/s
Q5	3,070	ft^3/s	7,830	ft^3/s
Q10	3,860	ft^3/s	9,890	ft^3/s
Q25	4,950	ft^3/s	13,000	ft^3/s
Q50	5,840	ft^3/s	15,700	ft^3/s
Q100	6,820	ft^3/s	18,900	ft^3/s
Q200	8,015	ft^3/s	22,400	ft^3/s
Q500	9,410	ft^3/s	28,000	ft^3/s

Reported by: Jeff DeGraff
Date: October 18, 2018

Note: All elevations based on North American Vertical Datum (NAVD) of 1988.

HYDRAULIC REPORT

The software package that was used to develop the SRH-2D (Sediment and River Hydraulics, Two-Dimensional) model for the existing and proposed crossings is Aquaveo's Surface-water Modeling System (SMS) Version 12.3. The program allows the user to develop a two-dimensional (2D) hydraulic, sediment and temperature model that incorporates the Finite Volume method in conjunction with implicit first- and second-order numerical schemes to approximate a solution for the 2D depth averaged Saint Venant equations.

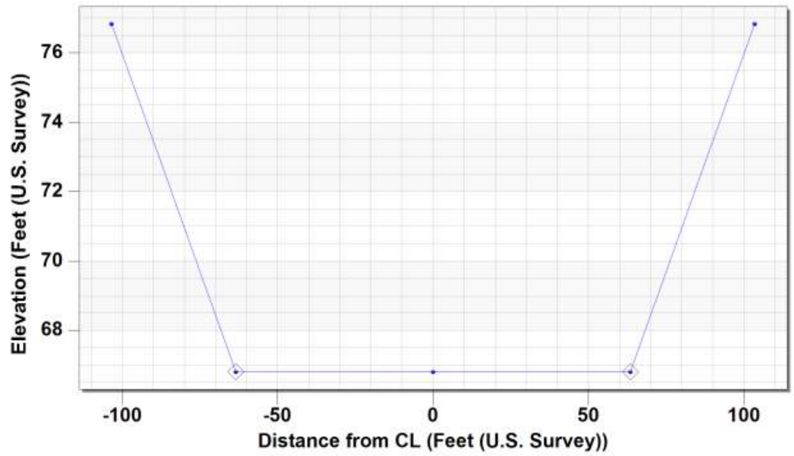
Field measurements were not taken for this project but USGS Water-Resources Investigation Report 97-4189, titled "Flood of October 1996 in Southern Maine" was utilized for calibration purposes.

Surface Generation

Conventional survey was completed for this project and incorporated as a DTM file. This file encompassed stream bathymetry but did not adequately encompass the floodplain.

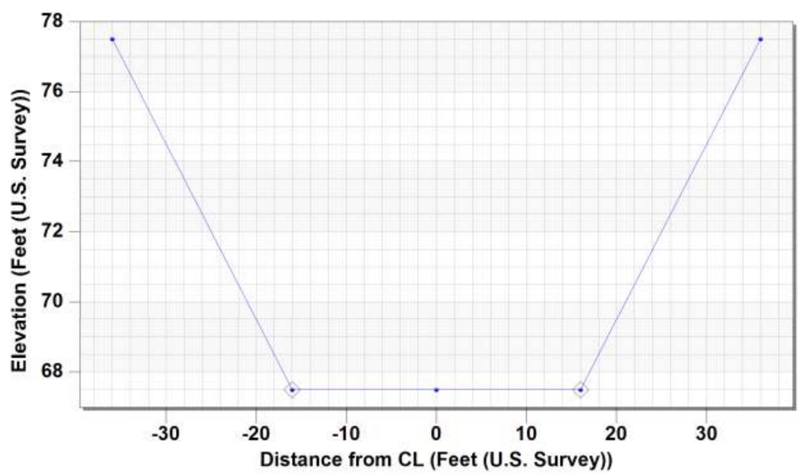
a. A LiDAR surface was used to supplement the on-the-ground survey to incorporate missing floodplain areas.
b. DEM/LiDAR panels were downloaded from the NOAA Data Access Viewer Website. https://coast.noaa.gov/dataviewer/#/LiDAR/search/
b. The geometric data in SMS included:
    i. Survey data.
    ii. LiDAR data.
c. SMS's feature stamping tool was used to generate additional bathymetry.
    i. Presumpscot River
        - Slope was determined using information regarding the Mallison Falls Dam on the Low Impact Hydropower's institute website and existing topography/bathymetry at the Saccarappa Dam provided by Sappi (Dam Owner)
        - Tailwater elevation 71-ft (NGVD29) taken from Low Impact Hydropower Institute.
        - Invert elevation near Saccarappa Dam is approximately 66 ft +/- (NGVD29, Topo attached)
        - Slope: (71ft-66ft)/10464.48ft = 0.00478 ft/ft. For the purpose of this study, these elevations were used to define the Presumpscot River.
        - The following cross-section was assumed

Presumpscot River Cross-Section



ii. Little River
- Upstream of the crossing, a slope of 0.005 ft/ft is assumed
- Downstream of the crossing, the river bed is assumed to be relatively flat.
- The following cross-section is assumed

Little River Cross-Section



Mesh Development

a. The model domain was manually defined for both the existing and proposed conditions.

b. Triangular elements typically were used to model floodplain and tributaries.
c. Quadrilateral mesh elements typically were used to develop the mesh within the river.

Calibration Process

Since field measurements were not obtained, USGS Water-Resources Investigation Report 97-4189, titled “Flood of October 1996 in Southern Maine” (Report) was utilized for calibration purposes.

i. The Report documented peak water surface elevations at various sites on the Presumpscot and Little River. These water surface elevations were reported in NGVD29.
ii. The “Online Vertical Datum Transformation” website was used to determine the conversion from NGVD29 to NAVD88.
    - https://www.ngs.noaa.gov/cgi-bin/VERTCON/vert_con.prl
    - The difference in elevation from NGVD29 to NAVD88 at the Saccarappa Dam is -0.692ft.
    - The table below shows peak water surface elevations from the 1996 flood that were used to calibrate the model

Location	WSE (NGVD)	WSE (NAVD88)
Upstream of Saccarappa Dam	76.8 ft	76.1 ft
7 Rousseau Road, Windham	80.9 ft	80.21 ft
Upstream of Route 237 Bridge	88.6 ft	87.91 ft
Downstream of Route 237 Bridge	87.3 ft	86.61 ft
Mallison St. Bridge, Gorham (75.5 ft Downstream)	95.5 ft	94.81 ft

a. Downstream Boundary Condition
   iii. Based on the Report, approximately 1500-ft upstream of the dam, the peak water surface elevation of 76.1 ft was obtained and was used to define the tailwater elevation of the Presumpscot River.
   iv. Subcritical flow is anticipated. Therefore, the downstream boundary condition is an Exit-H type and the adjusted Water Surface Elevations are entered for different storm events.

b. Low Chord Elevation
   v. The superstructure is sloped but for this project the lowest value was used to define a constant low chord at this site. A low chord elevation of 84.11 ft, NAVD88 (84.78 ft, NGVD29)

Material Properties and Roughness Values

Initial values that produced good results.

Calibration Summary

Calibration Summary			
Location	Observed Value	Computed Value	Difference
DS 237	86.61	86.41	-0.20
US 237	87.91	87.60	-0.31
7 Rousseau Rd	80.21	79.89	-0.32

Rating Curve Development

a. The purpose of developing the rating curve for the Saccarappa Dam is to determine the water surface elevation upstream of the dam for various flows. The water surface elevation is used as the downstream boundary condition for the hydraulic models; this assumes that the water surface elevation is approximately the same 1500’ upstream of the dam.
b. The Engineering Manager, Barry Stemm, for the Saccarappa Dam provided the existing site plan for the dam. He also provided a top of spillway elevation of 70 ft +/- 0.2 ft (NGVD29).
c. The historic data provided in Table 7 of the USGS Water-Resources Investigation Report 97-4189 was used to determine the weir coefficient for the dam.
d. Assume dam efficiency is maintained below 12,400 cfs and decreases linearly above 12,400 cfs.
e. The rating curve is then developed using the broad crested weir equation. The rating curve is used to determine the water surface elevation of the desired flow event for the downstream boundary condition.

Check Existing Calibrated Model

a. The USGS Scientific Investigations Report 2009-5102, titled “Flood of April 2007 in Southern Maine” was utilized to check the calibrated model.
    i. This report documented peak water surface elevations at various site on the Presumpscot and Little River. These water surface elevations were reported in NAVD88.
    ii. The table below shows peak water surface elevations from the 2007 flood that were used to calibrate the model

Location	WSE (NAVD88)
Upstream of Route 237 Bridge	81.80 ft
Downstream of Route 237 Bridge	81.69 ft

b. Peak Flow during 2007
    i. A stream gage (01064118) is located downstream of the Saccarappa Dam. Data from the stream gage and National Weather Service information were used to develop a stage discharge graph.
    ii. The stage discharge graph was used to determine that the flow near the Saccarappa Dam during the 2007 flood was about 11,825 cfs.
c. Upstream Boundary Condition – Little River
    i. Two models were run to test the sensitivity of the water surface elevation by varying the flow in the Little River.
    ii. The first model used a flow based on streamstats upper limit confidence interval for the 2-year return event; approximately 2,040 cfs.
    iii. The second model used a flow based on streamstats upper limit confidence interval for the 25-year return event; approximately 4,950 cfs.
d. Additional Internal Boundary Conditions
    i. The Internal Sink Boundary Conditions were used to add flow to the model domain downstream of the Little River Confluence at the Mosher Brook and Inkhorn Brook confluences assuming a 2-year return per streamstats.
        - Mosher Brook – 68 cfs
        - Inkhorn Brook – 153 cfs
e. Upstream Boundary Condition – Presumpscot River
    i. The upstream boundary condition for the Presumpscot River was determined using mass conservation.
    ii. For the first model, the constant inflow had to be 9,564 cfs.
    iii. For the second model, the constant inflow had to be 6,654 cfs.
f. Downstream Boundary Condition
    i. Based on the 11,825 cfs flow and using the Saccarappa Dam rating curve, the water surface elevation was determined to be 72.99 ft.
    ii. Subcritical flow is anticipated for this event. Therefore, the downstream boundary condition is an Exit-H type.

Scour Analysis

a. In accordance with MaineDOT BDG 2.3.11.1
b. Analyze the Little River without the influence of the Saccarappa Dam
    iii. Normal slope of 0.00478, Composite n of 0.035
c. Neglect Mosher Brook and Inkhorn Brook for scour analysis
d. Largest Scour Potential will occur when there is minimal tailwater produced from Presumpscot
    i. Note: Based on the FIS and USGS April 2007 report, Presumpscot Dam is at peak flow and Little River is at low flow.
a. Design Event – 100yr
    i. Assume Presumpscot River is flowing at a 10-year event at Saccarappa Dam – 9,890 cfs
    ii. Little River Discharge – 6,820 cfs
    iii. US Presumpscot Discharge – 3,070 cfs

e. Check Event – 500yr
    i. Assume Presumpscot River is flowing at a 25-year event at Saccarappa Dam – 13,000 cfs
    ii. Little River Discharge – 9,410 cfs
    iii. US Presumpscot Discharge – 3,590

Estimated scour is based on procedures outlined in FHWA HEC 18, 5^{th} Edition, Publication No. FHWA-HIF-12-003, and the lab results of the soil samples taken at the site. Since the channel bed generally consists of clay and particle sizes were not determined, a sensitivity analysis was performed to capture the potential range in particle sizes associated between clay and silt and to estimate the critical shear stress. A particle range from 0.0003 mm to 0.03 mm was used. This corresponded to maximum scour elevations between EL. 52.7 and EL 49.6 respectively or scour depths between 9.8 feet and 12.9 feet.

SUMMARY (DOWNSTREAM DAM IN PLACE)

		Existing Structure	Recommended Structure
		100' Thru Girder	135' Plate Girder
Total Area of Waterway Opening	ft^2	1230	2030
Headwater elevation @ Q_2	ft	77.9	77.9
Headwater elevation @ Q_{50}	ft	83.8	83.8
Headwater elevation @ Q_{100}	ft	85.3	85.0
Headwater elevation @ Q_{500}	ft	89.0	88.5
Freeboard @ Q_{50}	ft	-0.1	1.6
Freeboard @ Q_{100}	ft	-1.3	0.4
Flood Of Record (October 1996) Elevation 87.91ft			
Outlet Velocity @ Q_2	ft/s	4.0	2.5
Outlet Velocity @ Q_{50}	ft/s	6.4	5.9
Outlet Velocity @ Q_{100}	ft/s	7.1	4.8
Outlet Velocity @ Q_{500}	ft/s	7.5	5.3

Reported by: Jeff DeGraff
Date: October 18, 2018

Note: All elevations based on North American Vertical Datum (NAVD) of 1988.

APPENDIX

Project Name:	Gorham	**PIN:**	23909.00
Stream Name:	Little River	**Town:**	Gorham
Bridge Name:	Little River Bridge	**Bridge No.**	3112
Route No.	ME 237	**USGS Quad:**	
Analysis by:	A. Mann	**Date:**	7/2/2018

Peak Flow Calculations by USGS Regression Equations (Hodgkins, 1999 & Lombard/Hodgkins, 2015)

Enter data in blue cells only!

	km^2	mi^2	ac	***Enter data in [mi^2]***
A	126.91	49.00	31360.0	Watershed Area *DRNAREA*
W	14.85	5.7	3669.1	Wetlands area (by NWI)
P_c	379109	4839661		watershed centroid (E, N; UTM 19N; meters)
County	Cumberland SE			*choose county from drop-down menu*
pptA	44.4			mean annual precipitation (inches; by look-up)
SG	0.18			sand & gravel aquifer as decimal fraction of watershed A

Worksheet prepared
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Environmental Office
Maine Dept. Transport
Augusta, ME 04333-00
207-557-1052
Charles.Hebson@mai
ver. 2016 Feb 05

A (km^2)	126.91	**Conf Lvl**	0.67
W (%)	11.70		NWI Wetlands % *STORNWI*

References:

Hodgkins, G.A., 1999.
Estimating the magnitude of peak flows for streams
in Maine for selected recurrence intervals
WRIR 99-4008, USGS Augusta, ME

Ret Pd T (yr)	Peak Flow Estimate Lower	Q_T (m^3/s)	Upper	Q_T (ft^3/s)
1.1		16.77		592.0
2		31.91		1126.8
5		47.69		1683.8
10		59.10		2086.8
25		74.06		2615.1
50		85.62		3023.2
100		97.91		3457.0
500		127.77		4511.5

Lombard, P.J. & G.A. Hodgkins, 2015.
Peak flow regression equations for small, ungaged streams
Maine - Comparing map-based to field-based variables
SIR 2015-4059, USGS, Augusta, ME

$Q_T = b \times A^a \times 10^{-wW}$

µ (log)	1.50
s (log)	2.093E-01

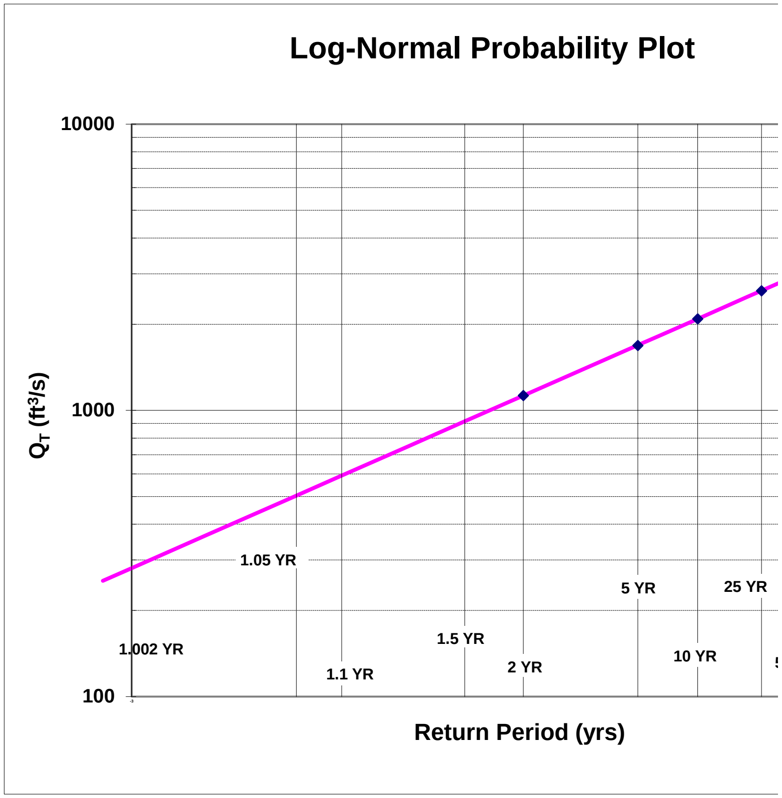
Flood Frequency Curve

Ret Pd T	Cum Prob F	Q_T (m3/s usgs)	Log-Normal Distn norm std var z	log (Q m3/s)	Q (m^3/s)	Q (ft^3/s)	log (Q ft3/s)	Q_T (ft3/s usgs)
1.001	0.001		-3.091	0.86	7.19	253.87	2.40	
1.002	0.002		-2.879	0.90	7.97	281.15	2.45	
1.010	0.010		-2.330	1.02	10.38	366.27	2.56	
1.050	0.048		-1.668	1.15	14.28	503.87	2.70	
1.100	0.091		-1.335	1.22	16.77	591.66	2.77	
1.250	0.200		-0.842	1.33	21.27	750.57	2.88	
1.500	0.333		-0.431	1.41	25.93	914.97	2.96	
1.750	0.429		-0.180	1.47	29.26	1032.50	3.01	
2	0.500	31.91	0.000	1.50	31.91	1126.09	3.05	1126.09
5	0.800	47.69	0.842	1.68	47.88	1689.49	3.23	1682.70
6.300	0.841		1.000	1.71	51.67	1823.25	3.26	
10	0.900	59.10	1.282	1.77	59.19	2088.57	3.32	2085.45
20	0.950		1.645	1.85	70.51	2488.30	3.40	
25	0.960	74.06	1.751	1.87	74.21	2618.53	3.42	2613.45
50	0.980	85.62	2.054	1.93	85.88	3030.39	3.48	3021.24
100	0.990	97.91	2.326	1.99	97.94	3455.92	3.54	3454.84
200	0.995		2.576	2.04	110.45	3897.53	3.59	
500	0.998	127.77	2.878	2.11	127.78	4508.99	3.65	4508.63
1000	0.999		3.090	2.15	141.53	4994.28	3.70	

Adjust Q_T Vertical Plot Scale to Contain FF Curve

"right click" QT scale and set min, max scale values

	Min $Q_{1.002}$	Max Q_{500}
Q	281.15	4508.99
log(Q)	2.45	3.65
scale Q	*100.00*	*10000.00*



Project Name:	Gorham		**PIN:**	23909.00
Stream Name:	Little River		**Town:**	Gorham
Bridge Name:	Little River Bridge		**Bridge No.**	3112
Route No.	ME 237		**USGS Quad:**	
Analysis by:	A. Mann		**Date:**	7/2/2018

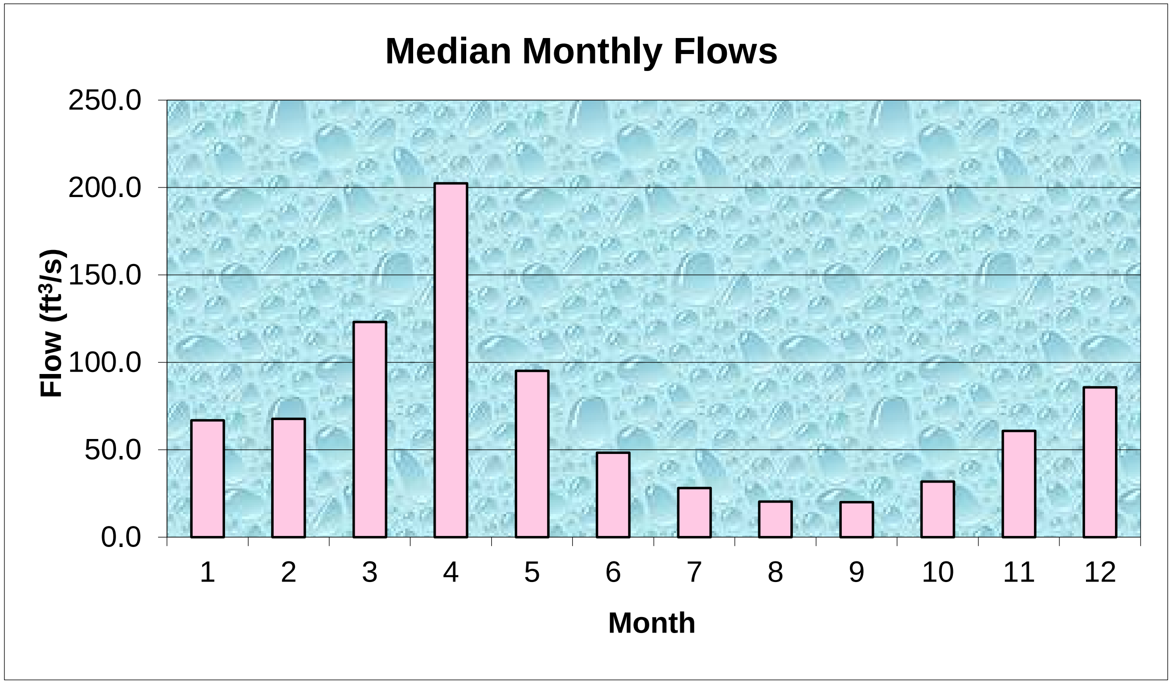
DO NOT ENTER ANY DATA ON THIS PAGE; EVERYTHING IS CALCULATED

MAINE MONTHLY MEDIAN FLOWS and HYDRAULIC GEOMETRY BY USGS REGRESSION EQUATIONS (2004)

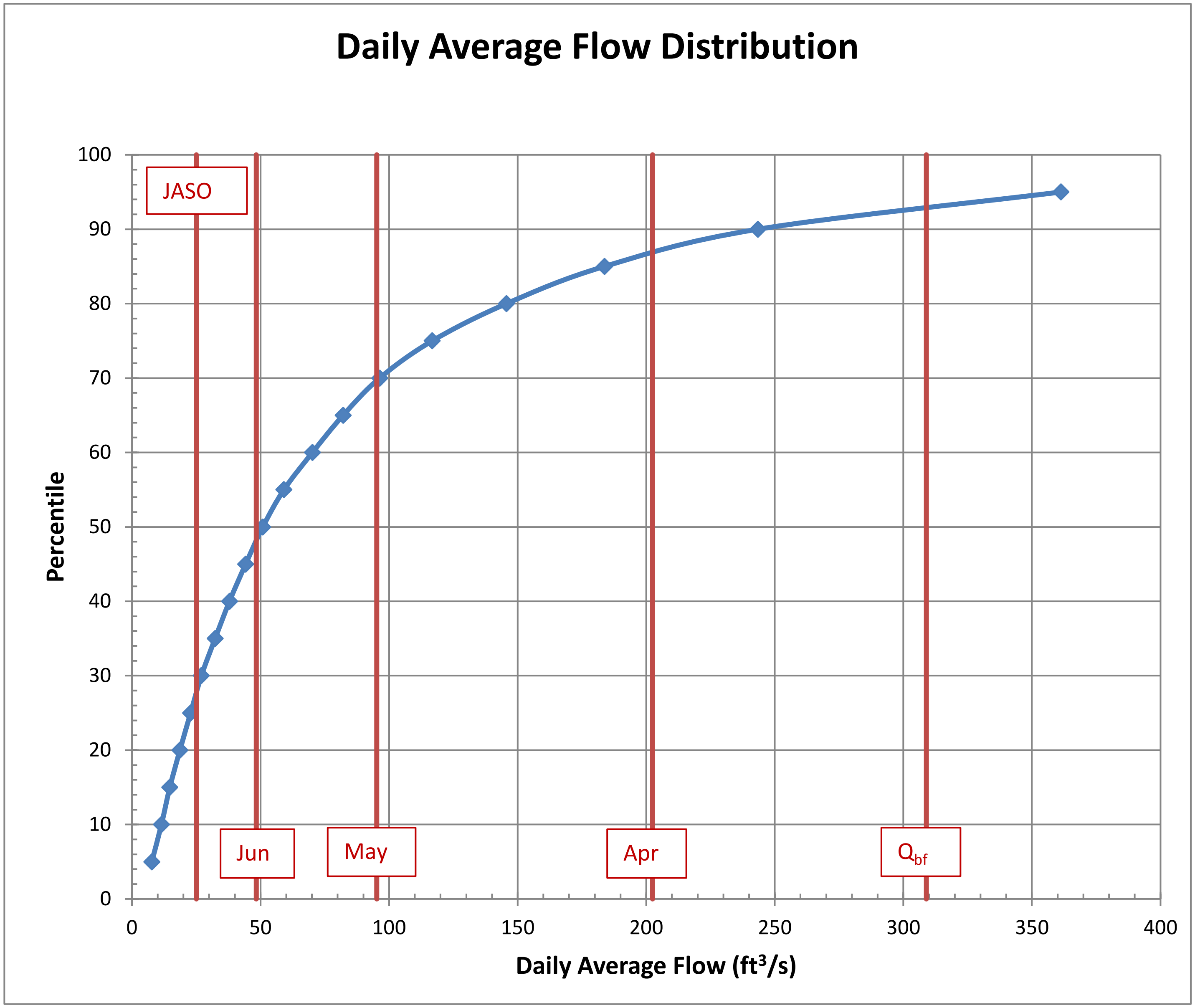
	Value	Variable	Explanation
	49.00	*A*	Area (mi^2)
379109	4839661	P_c	Watershed centroid (E,N; UTM; Zone 19; meters)
	42.73	*DIST*	Distance from Coastal reference line (mi)
	44.4	*pptA*	Mean Annual Precipitation (inches)
	0.18	*SG*	Sand & Gravel Aquifer (decimal fraction of watershed area)

Month	Q_{median} (ft3/s)	(m^3/s)
Jan	**66.77**	1.8922
Feb	**67.66**	1.9173
Mar	**123.11**	3.4887
Apr	**202.45**	5.7372
May	**95.19**	2.6976
Jun	**48.34**	1.3700
Jul	**28.04**	0.7948
Aug	**20.45**	0.5795
Sep	**19.97**	0.5658
Oct	**31.78**	0.9007
Nov	**60.82**	1.7235
Dec	**85.66**	2.4276

Q_{bf}	**308.9**	
ann avg	**98.0**	
ann med	**51.5**	
$Q_{1.002}$	**281.2**	
$Q_{1.01}$	**366.3**	
$Q_{1.05}$	**503.9**	
Q_{bf}	**517.9**	assume v = 4ft/s



References



Daily Avg Flow Dist

A_{ws} = (mi^2) 49.0

Q (ft^3/s)

Pctl	Median	84^{th} pctl
5	7.71	12.41
10	11.46	17.23
15	14.73	21.51
20	18.65	26.08
25	22.81	30.58
30	27.00	34.82
35	32.34	39.80
40	37.93	45.77
45	44.21	51.75
50	50.90	61.10
55	59.11	71.11
60	70.20	83.48
65	82.13	97.25
70	96.33	113.46
75	116.78	136.45
80	145.64	162.91
85	183.76	208.77
90	243.42	280.33
95	361.26	435.94

Q_{bf}	308.9
$Q_{1.002}$	281.2
$Q_{1.1}$	592.0
Q_2	1126.8

Normalized Distribution

Q/A_{ws} {$(ft^3/s)/mi^2$}

Pctl	Median	84th pctl
5	0.157	0.253
10	0.234	0.352
15	0.301	0.439
20	0.381	0.532
25	0.466	0.624
30	0.551	0.711
35	0.660	0.812
40	0.774	0.934
45	0.902	1.056
50	1.039	1.247
55	1.206	1.451
60	1.433	1.704
65	1.676	1.985
70	1.966	2.316
75	2.383	2.785
80	2.972	3.325
85	3.750	4.261
90	4.968	5.721
95	7.373	8.897

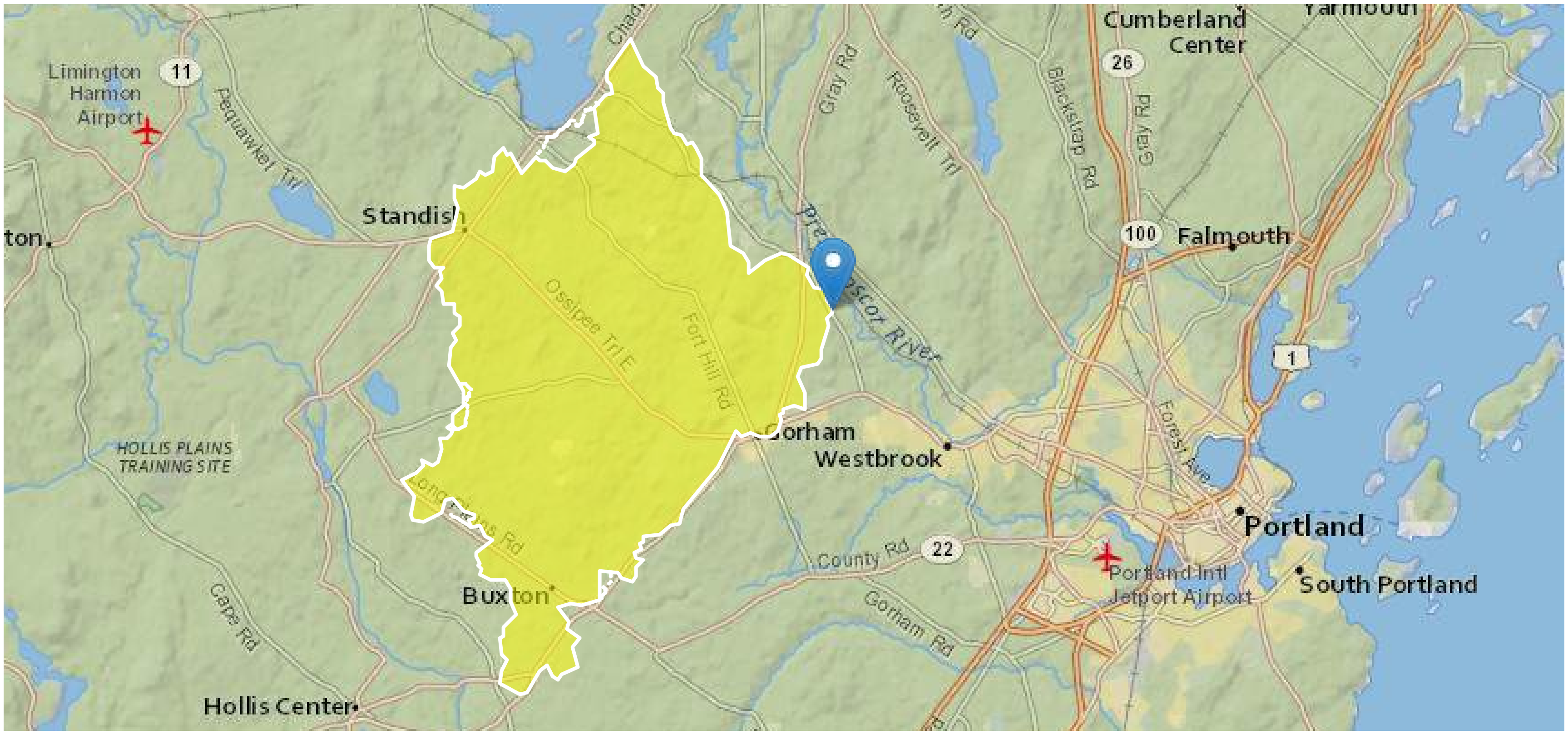
Monthly Median Flows

Mar	123.11	
	123.11	
Apr	202.45	0.00
	202.45	100.00
May	95.19	0.00
	95.19	100.00
Jun	48.34	0.00
	48.34	100.00
JASO	25.06	0.00
	25.06	100.00
Qbf	308.94	0.00
	308.94	100.00

<=== Print Graph & Table within Border
Scale Horizontal Axis Appropriately

Gorham, Rt 237 Bridge over the Little River BR 3112 WIN 23909.00

Region ID: ME
Workspace ID: ME20180702192204625000
Clicked Point (Latitude, Longitude): 43.71358, -70.41372
Time: 2018-07-02 15:22:16 -0400



Much of the area near Sebago lake is sand and there is a significant closed basin with ponds in this area.

Basin Characteristics

Parameter Code	Parameter Description	Value	Unit
DRNAREA	Area that drains to a point on a stream	49	square miles
STORNWI	Percentage of strorage (combined water bodies and wetlands) from the Nationa Wetlands Inventory	11.7	percent
BSLDEM10M	Mean basin slope computed from 10 m DEM	5.13	percent
CENTROIDX	Basin centroid horizontal (x) location in state plane coordinates	379109.37	feet
CENTROIDY	Basin centroid vertical (y) location in state plane units	4839661.09	feet
COASTDIST	Shortest distance from the coastline to the basin centroid	43	miles
ELEV	Mean Basin Elevation	229.6	feet
ELEVMAX	Maximum basin elevation	531.3	feet
LC06WATER	Percent of open water, class 11, from NLCD 2006	0.05	percent
LC11DEV	Percentage of developed (urban) land from NLCD 2011 classes 21-24	12.7	percent
LC11IMP	Average percentage of impervious area determined from NLCD 2011 impervious dataset	2.72	percent
PRECIP	Mean Annual Precipitation	45.3	inches
SANDGRAVAF	Fraction of land surface underlain by sand and gravel aquifers	0.18	dimensionless
SANDGRAVAP	Percentage of land surface underlain by sand and gravel aquifers	17.99	percent
STATSGOA	Percentage of area of Hydrologic Soil Type A from STATSGO	32.3	percent

General Disclaimers

This watershed has been edited, computed flows may not apply.

Bankfull Statistics Parameters [Central and Coastal Bankfull 2004 5042]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	49	square miles	2.92	298

Bankfull Statistics Flow Report [Central and Coastal Bankfull 2004 5042]

Statistic	Value	Unit
Bankfull Streamflow	309	ft^3/s
Bankfull Width	58	ft
Bankfull Depth	2.23	ft
Bankfull Area	129	ft^2

Bankfull Statistics Citations

Dudley, R.W.,2004, Hydraulic-Geometry Relations for Rivers in Coastal and Central Maine: U.S. Geological Survey Scientific Investigations Report 2004-5042, 30 p (http://pubs.usgs.gov/sir/2004/5042/pdf/sir2004-5042.pdf)

Peak-Flow Statistics Parameters [Statewide Peak Flow Full GT 12sqmi WRI 99 4008]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	49	square miles	0.93	1653
STORNWI	Percentage of Storage from NWI	11.7	percent	0.7	26.7

Peak-Flow Statistics Flow Report [Statewide Peak Flow Full GT 12sqmi WRI 99 4008]

PII: Prediction Interval-Lower, PIu: Prediction Interval-Upper, SEp: Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unit	PII	PIu	SE	SEp	Equiv. Yrs.

Statistic	Value	Unit	PII	PIu	SE	SEp	Equiv. Yrs.
2 Year Peak Flood	1130	ft^3/s	630	2020	35.1	35.1	1.8
5 Year Peak Flood	1680	ft^3/s	933	3040	36.1	36.1	2.5
10 Year Peak Flood	2090	ft^3/s	1140	3820	36.8	36.8	3.2
25 Year Peak Flood	2620	ft^3/s	1390	4910	38.6	38.6	4.1
50 Year Peak Flood	3020	ft^3/s	1580	5790	39.9	39.9	4.8
100 Year Peak Flood	3460	ft^3/s	1770	6770	41.2	41.2	5.4
500 Year Peak Flood	4510	ft^3/s	2180	9340	44.9	44.9	6.4

Peak-Flow Statistics Citations

Hodgkins, G. A.,1999, Estimating the Magnitude of Peak Flows for Streams in Maine for Selected Recurrence Intervals: U.S. Geological Survey Water-Resources Investigations Report 99-4008, 45 p. (http://me.water.usgs.gov/99-4008.pdf)

Annual Flow Statistics Parameters [Statewide Annual SIR 2015 5151]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	49	square miles	14.9	1419
SANDGRAVAF	Fraction of Sand and Gravel Aquifers	0.18	dimensionless	0	0.212
ELEV	Mean Basin Elevation	229.6	feet	239	2120

Annual Flow Statistics Disclaimers [Statewide Annual SIR 2015 5151]

One or more of the parameters is outside the suggested range. Estimates were extrapolated with unknown errors

Annual Flow Statistics Flow Report [Statewide Annual SIR 2015 5151]

Statistic	Value	Unit
Mean Annual Flow	106	ft^3/s

Annual Flow Statistics Citations

Dudley, R.W.,2015, Regression equations for monthly and annual mean and selected percentile streamflows for ungaged rivers in Maine: U.S. Geological Survey Scientific Investigations Report 2015–5151, 35 p. (http://dx.doi.org/10.3133/sir20155151)

Application Version: 4.2.1

Flood of October 1996 in Southern Maine

by Glenn Hodgkins and Gregory J. Stewart

U.S. DEPARTMENT OF THE INTERIOR
U.S. GEOLOGICAL SURVEY
Water-Resources Investigations Report 97-4189

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FEDERAL EMERGENCY MANAGEMENT AGENCY

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Augusta, Maine
1997

U.S. DEPARTMENT OF THE INTERIOR

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FIGURES

TABLES

CONVERSION FACTORS

Multiply	By	To obtain
Length		
inch (in)	25.40	millimeter
foot (ft)	0.3048	meter
mile (mi)	1.609	kilometer
Area		
square mile (mi^2)	2.590	square kilometer
Volume		
cubic foot (ft^3)	0.02832	cubic meter
Flow		
cubic foot per second (ft^3/s)	0.02832	cubic meter per second

VERTICAL DATUM

In this report, all elevations are referenced to the National Geodetic Vertical Datum of 1929, a geodetic datum derived from a general adjustment of the first-order level nets of both the United States and Canada. It was formerly called Sea Level Datum of 1929.

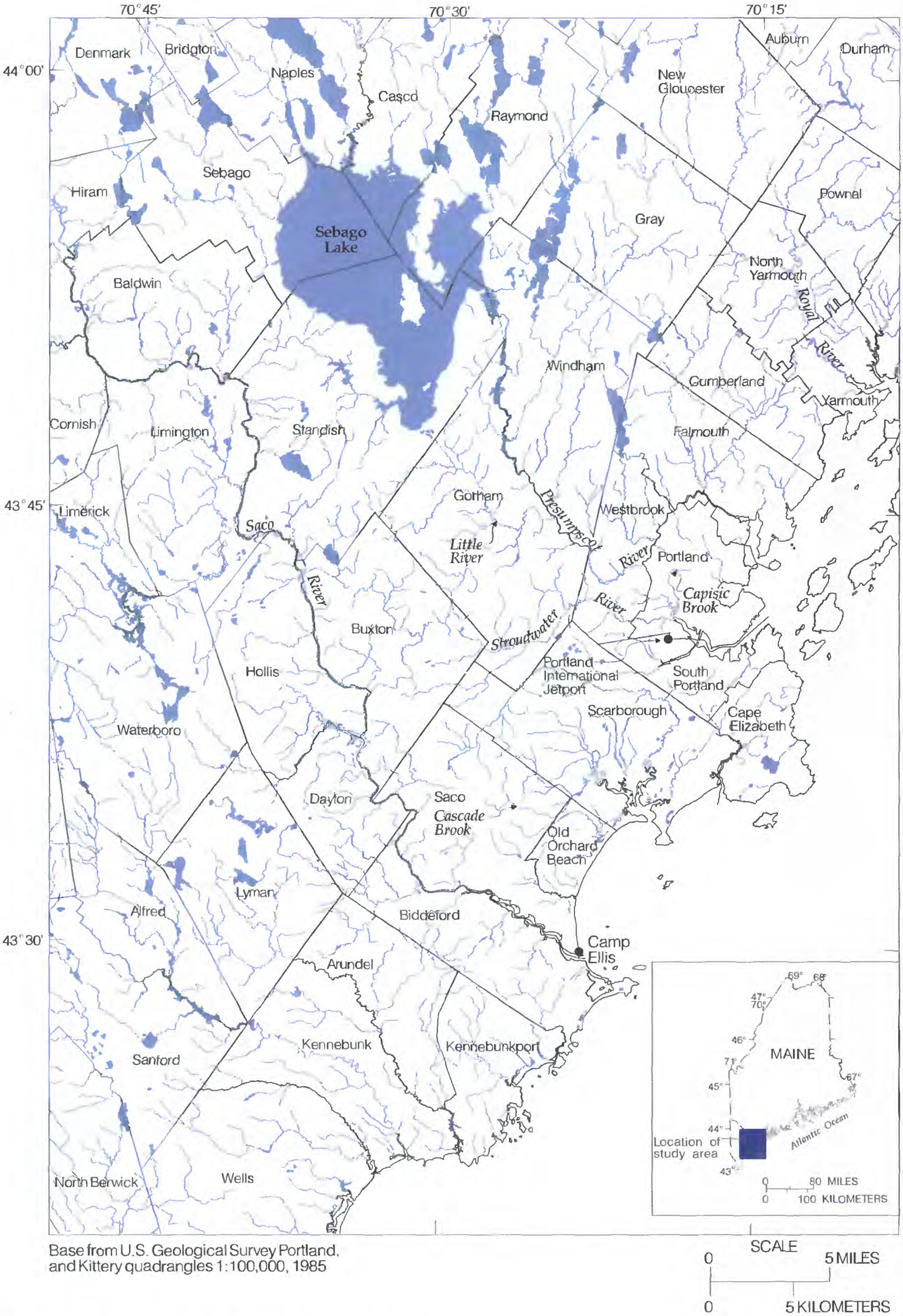

Figure 1. Location of study area and selected points.

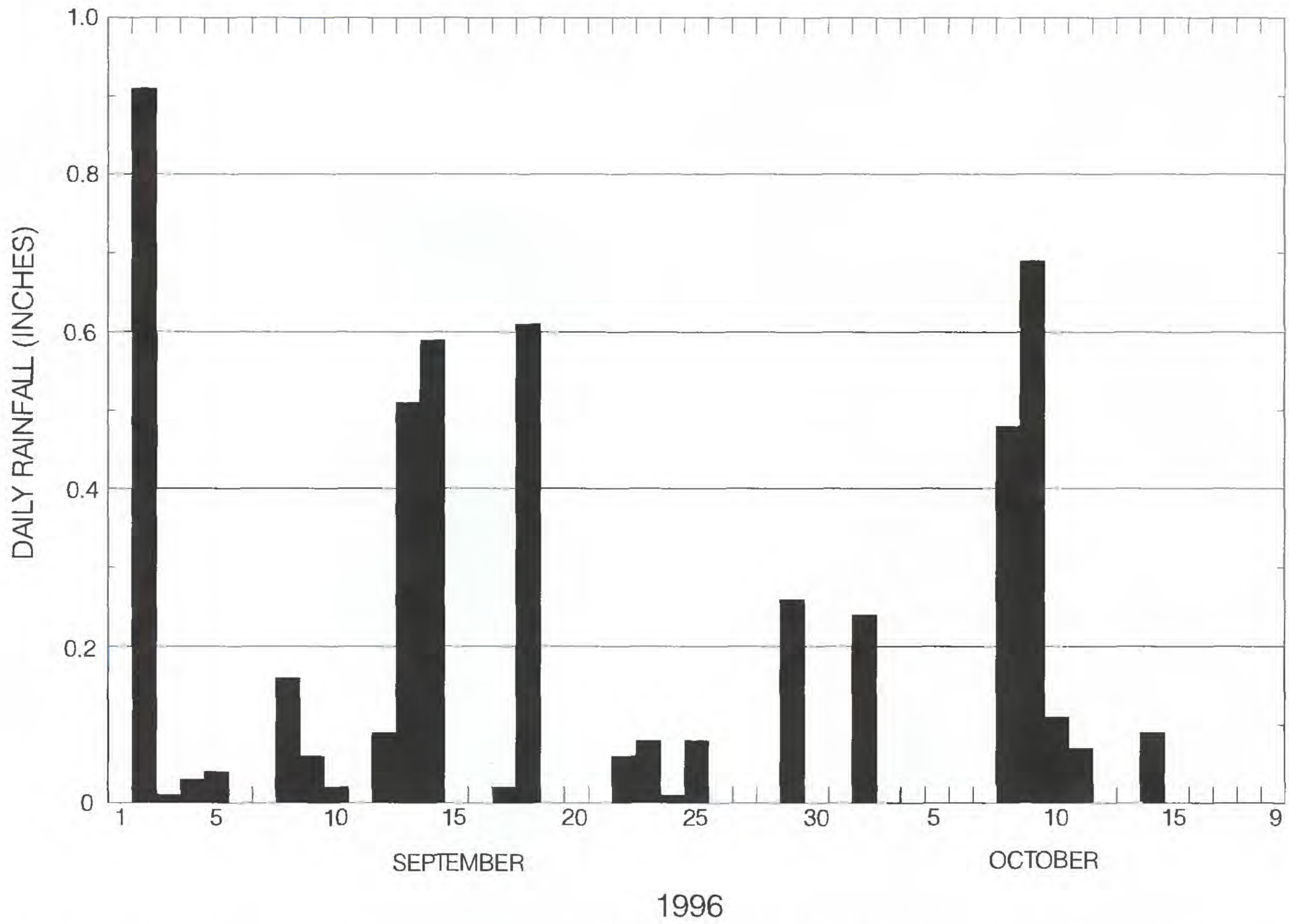

Figure 2. Daily rainfall totals from September 1, 1996 to October 19, 1996 at Portland International Jetport (National Oceanic and Atmospheric Administration, 1996a).

the northeaster. This "conveyor belt" of moisture from Hurricane Lili, combined with the slow movement of the northeaster, resulted in the tremendous rainfall totals in Maine (Tom Hawley, National Weather Service, written commun., 1996).

Rainfall Amounts

The rainfall totals from the October 20-22, 1996 storm are shown with lines of equal rainfall (rainfall isolines) in figure 9 (page 10). These isolines were computed by the National Weather Service (NWS), based on rainfall totals from NWS observers in southern Maine. The largest recorded totals were: 19.19 inches in the Camp Ellis area of Saco, 19.00 inches in Gorham, 17.62 inches in Westbrook, 16.00 inches in South Windham, and 15.52 inches in Scarborough (Tom Hawley, written commun., 1997).

The National Weather Service rain gage at the Portland International Jetport was the only recording rain gage in southern Maine during the October 20-22, 1996 storm. Figure 3 is a graph of the cumulative rainfall amounts at the Jetport from October 20-22, 1996 and is based on hourly rainfall data from the Jetport gage (National Oceanic and Atmospheric Administration, 1996b). Maximum rainfall totals for different time periods were calculated using this same hourly rainfall data. The maximum 48-hour, 24-hour, 12-hour, 6-hour, and 2-hour rainfall totals at the Jetport were 14.65 inches, 13.32 inches, 8.86 inches, 4.81 inches, and 2.06 inches, respectively.

The data in figure 3 show that most of the rain at the Jetport from this storm fell in a 26-hour period from October 20 at 8:00 p.m. to October 21 at 10:00 p.m. The relatively uniform slope in this figure indicates that most of the rain at the Jetport fell at a relatively constant rate.

At the Jetport, the total rainfall for October 20-22, 1996 was 14.65 inches and the maximum 24-hour total was 13.32 inches. This 24-hour total shattered the previous 24-hour record of 7.83 inches set during Hurricane Bob in August 1991. Rainfall records for Portland have been kept since 1871 (Tom Hawley, oral commun., 1997). The total rainfall isolines in figure 9, indicate that the 13.32-inch 24-hour rainfall at the Jetport was exceeded at other locations.

Rainfall Frequency

The Atlas of Precipitation Extremes for the Northeastern United States and Southeastern Canada (Wilks and Cember, 1993) contains maps with recurrence-interval isolines for 1-day and 2-day rainfall totals. The n-year (where n equals 100, 50, 25, 10, 5, and 2), 1-day and 2-day rainfall recurrence intervals for Portland were interpolated from these maps. The recurrence interval is the *average* period of time between rainfalls that are greater than, or equal to, a specified magnitude. As an example of recurrence interval, the 100-year rainfall is the rainfall that, on long-term average, would be equaled or exceeded once every 100 years. Conversely, this means that there is a one percent chance, every year, that a rainfall of this magnitude will be equaled or exceeded.

The 48-hour, n-year rainfall amounts for Portland were computed by applying the adjustment factor from the Atlas of Precipitation Extremes for the Northeastern United States and Southeastern Canada to the 2-day rainfall totals. The 24-hour rainfalls were computed by applying the adjustment factor to the 1-day rainfalls.

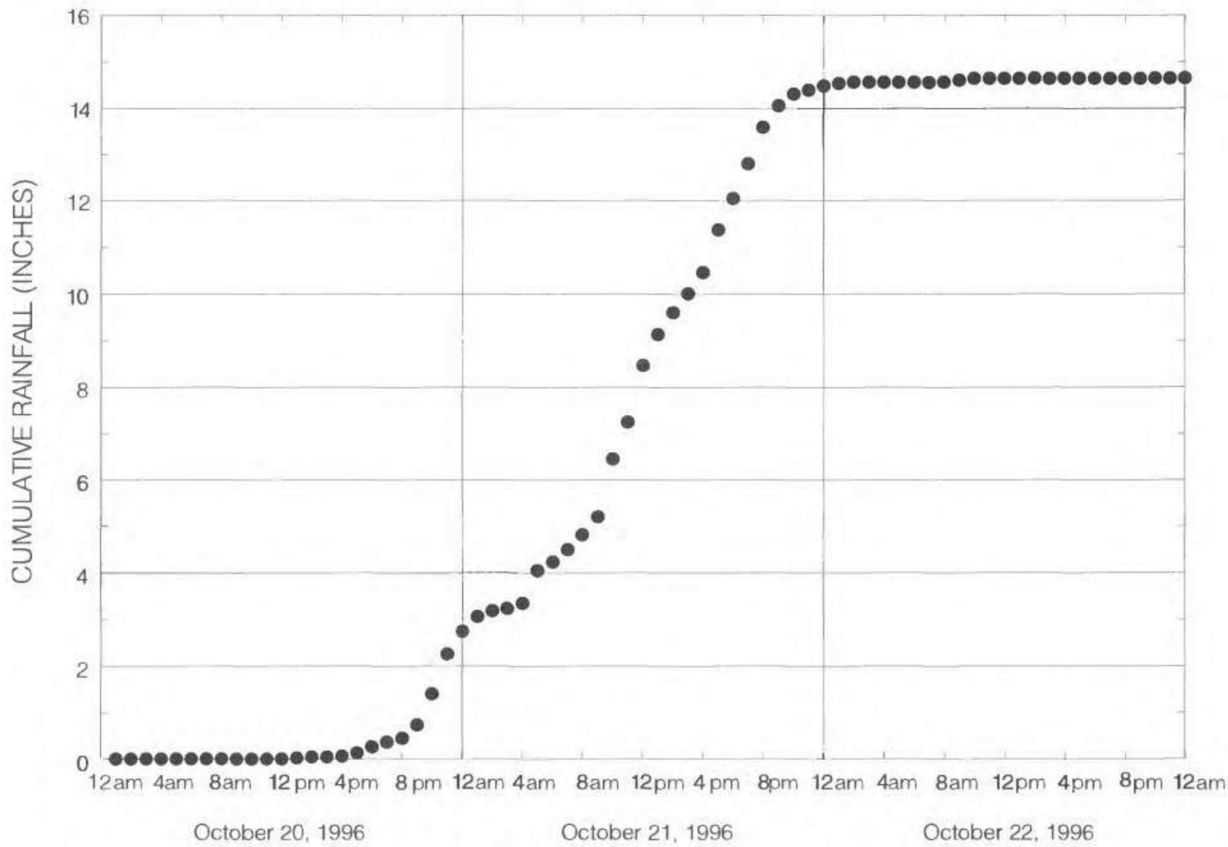

Figure 3. Cumulative rainfall from October 20-22, 1996 at Portland International Jetport.

Table 1. Peak water-surface elevations for October 1996 southern Maine flood
[na indicates not applicable]

Site number	Location	Latitude (degrees, minutes, seconds)	Longitude (degrees, minutes, seconds)	Upstream elevation[a] (feet)	Downstream elevation[a] (feet)	Elevation[b] (feet)
	Presumpscot River					
1	Hurricane Rd. Bridge, Windham/Gorham	43 45 58	70 26 55	138.3 (50 ft)	None found.	na
2	Gambo Rd. Bridge (discon-tinued)[c], Windham/Gorham	43 44 51	70 26 21	137.2 (20 ft)	118.0 (500 ft)	na
3	Route 202 Bridge[d], Windham/Gorham	43 44 02	70 25 35	115.7 (600 ft)	97.6 (300 ft)	na
4	Mallison St. Bridge[e], Windham/Gorham	43 43 40	70 25 15	None found.	95.5 (75 ft) 87.5 (900 ft)	na
5	7 Rousseau Road, Windham	43 41 55	70 23 11	na	na	80.9
6	40 Lincoln Street[f], Westbrook	43 40 52	70 22 19	na	na	76.8
7	Bridge St. Bridge, Westbrook	43 40 39	70 22 03	58.8 (130 ft)	55.5 (75 ft)	na
8	20 Water St., Westbrook	43 40 50	70 21 29	na	na	54.5
9	Cumberland St. Bridges[g], Westbrook	43 40 59	70 21 08	52.5 (10 ft)	51.6 (30 ft)	na
10	S. D. Warren Bridge, Westbrook	43 41 11	70 20 55	47.5 (5 ft)	None found.	na
11	Route 302 Bridge, Westbrook/Portland	43 42 11	70 19 32	43.8 (50 ft)	None found.	na
12	Blackstrap Rd. Bridge, Falmouth	43 43 28	70 18 11	41.0 (50 ft)	40.0 (200 ft)	na
13	Route 26/100 Bridge, Falmouth	43 43 42	70 17 45	38.8 (100 ft)	None found.	na
14	I-95 Bridges, Falmouth	43 43 40	70 17 10	37.2 (200 ft)[h]	36.4 (150 ft)[i]	na
15	Smelt Hill Dam, Falmouth	43 43 06	70 16 11	26.5 (350 ft)	14.4 (170 ft)	na

Table 1. Peak water-surface elevations for October 1996 southern Maine flood--*Continued*
[na indicates not applicable]

Site number	Location	Latitude (degrees, minutes, seconds)	Longitude (degrees, minutes, seconds)	Upstream elevation[a] (feet)	Downstream elevation[a] (feet)	Elevation[b] (feet)
	Casco/Naples					
41	Crooked River Bridge, Route 11	43 58 45	70 33 51	None found.	280.8 (5 ft)	na
	Cornish					
42	Pease Brook Culvert, Route 25/117	43 47 15	70 45 57	326.4 (30 ft)	323.7 (30 ft)	na
	Cumberland					
43	East Branch Piscataqua River Culvert, Route 9	43 46 27	70 15 09	44.5 (55 ft)	44.1 (105 ft)	na
	Falmouth					
44	Piscataqua River Culverts, Leighton Rd.	43 44 15	70 17 40	38.2 (20 ft)	38.1 (85 ft)	na
	Gorham					
45	Little River Bridge, Route 202	43 43 15	70 25 28	92.5 (60 ft)	92.3 (50 ft)	na
46	Little River Bridge, Route 237	43 42 48	70 24 52	88.6 (120 ft)	87.3 (100 ft)	na
47	South Branch Stroudwater River Culvert, Hodgdon Rd.	43 37 42	70 26 55	None found.	103.9 (100 ft)	na
48	Tannery Brook Culverts, Route 114	43 41 11	70 26 51	147.1 (160 ft)	140.8 (50 ft)	na
	Gray					
49	Collyer Brook Bridges, Route 202/100/4	43 55 05	70 19 02	None found.	182.9 (50 ft)[c]	na

Table 2. Peak-flow data for October 1996 flood and previous floods in southern Maine
[dash indicates not available, none indicates not a current or former U.S. Geological Survey stream gaging station, cubic feet per second abbreviated as ft^3/s, square miles abbreviated as mi^2, > indicates greater than, < indicates less than]

			Peak flow during October 1996 flood			Highest peak flow previously known			
Site number[a]	USGS gaging station number	Stream and location	Flow (ft^3/s)	Recurrence interval	Exceedance probability	Year	Flow (ft^3/s)	Period of known peak flows	Drainage area (mi^2)
10	01064118	Presumpscot River at S. D. Warren Bridge, Westbrook	23,300[b]	250 years[c]	0.004	1991	13,900	1895-present	577[d]
14	None	Presumpscot River at I-95, Falmouth	31,500	-	-	-	-	-	640[e]
28	01064158	Stroudwater River at Congress St., Portland	13,400	>500 years[c]	<0.002	1977	1,160	1975-1977 (seasonal)	27.5
33	None	Cascade Brook at Cascade Falls, Saco	1,650	>500 years[c]	<0.002	-	-	-	3.13
41	01063100	Crooked River at Route 11, Casco/ Naples	2,810	4 years	0.250	1996	2,800	1975-1977 (seasonal), 1995-present	150
42	01066100	Pease Brook at Route 25/117, Cornish	352	10 years	0.100	1969	486	1964-1974	4.80
49	01059800	Collyer Brook at Route 202/100/4, Gray	1,150	15 years	0.067	1969	1,220	1964-1982	13.8
50	01066500	Little Ossipee River, at Nason's Mills Rd., Limington	5,800	30 years	0.033	1936	8,530	1936, 1940-1982	168
55	None	Capisic Brook at Capisic St., Portland	865	>500 years[c]	<0.002	-	-	-	2.63
59	01069500	Mousam River at Wichers Mill Rd., Sanford	3,600	50 years	0.020	1983	4,020	1939-1984	99.0
62	01063310	Stony Brook at Route 114, Sebago	106	7 years	0.143	1996	63	1995-present	1.50
68	01069700	Branch Brook at Route 9A, Wells	1,020	40 years	0.025	1972	723	1964-1974	10.3
74	01060000	Royal River at East Main St., Yarmouth	4,410	3 years	0.330	1977	11,500	1949-present	141

[a] The site number refers to the site numbers from Table 1.
[b] Based on a drainage area correction from site 14.
[c] Recurrence intervals greater than 100 years have a large uncertainty associated with them.
[d] The drainage area is 136 mi^2 below the outlet of Sebago Lake.
[e] The drainage area is 199 mi^2 below the outlet of Sebago Lake.

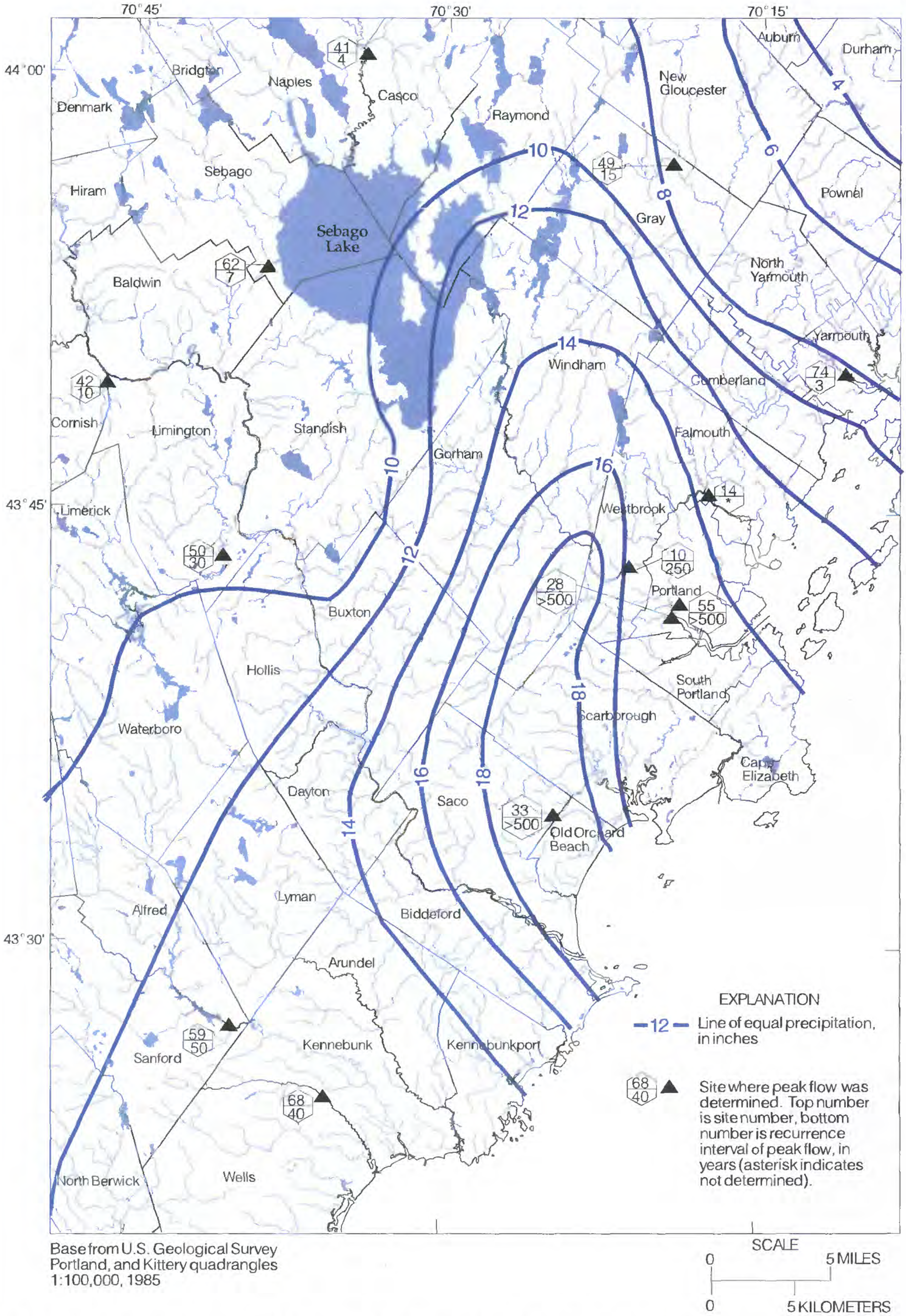

Figure 10. Total rainfall isolines and location of sites where peak flows and recurrence intervals were determined for October 1996 southern Maine flood. [Rainfall isolines provided by National Weather Service.]

Peak-Flow Frequency

The recurrence interval is the *average* period of time between peak flows that are greater than, or equal to, a specified magnitude. For example, the 50-year peak flow is the flow that would be exceeded or equaled, on long-term average, once in 50 years. This does not imply that flooding will happen at regular intervals. Two 50-year peak flows could be experienced in 2 consecutive years. Conversely, a 50-year peak flow might not be experienced for 100 years or longer. The reciprocal of the recurrence interval is called the annual exceedance probability; that is, the probability that a given peak flow will be exceeded or equaled in any given year. For example, the annual exceedance probability of the 50-year peak flow would be 0.02. In other words, there is a 2 percent chance that the 50-year peak flow will be exceeded or equaled in any given year.

The 500-, 100-, 50-, 25-, 10-, 5-, and 2-year recurrence-interval peak flows for sites in table 2 were determined as explained below. The peak flows from the October 1996 flood were then compared to these recurrence-interval peak flows to determine the recurrence intervals of the October 1996 peak flows.

All recurrence interval estimates have an uncertainty associated with them. The uncertainty generally increases as the recurrence interval increases. Recurrence intervals that are greater than 100 years have a large amount of uncertainty associated with them. For example, the calculated 500-year peak flow for Cascade Brook in Saco (site 33) is 650 ft^3/s, but the one-standard-error-of-estimate range of the 500-year peak flow extends from about 200 ft^3/s to about 1200 ft^3/s. This means that the actual 500-year peak flow will be between 200 ft^3/s and 1200 ft^3/s for about two-thirds of sites with the same drainage basin characteristics (drainage area, stream slope, etc.) as site 33. This corresponds to a flow uncertainty factor of about 2 to 3. The recurrence interval uncertainty is greater than the flow uncertainty. The recurrence interval for a given flow of 650 ft^3/s at site 33 is estimated to be 500 years with a recurrence interval uncertainty factor of about 9 (William H. Kirby, U.S. Geological Survey, written commun., 1997). That is, at two-thirds of sites similar to site 33, the actual recurrence interval is estimated to be in the range of 60 to 4500 years. Given these uncertainties, a reasonable estimate of the recurrence interval of the 1650 ft^3/s October 1996 peak flow at site 33 is not possible. In general, because of the extreme uncertainties in peak-flow recurrence intervals greater than 500 years, applicable peak-flow recurrence intervals from the October 1996 flood are reported as >500 years (in table 2), instead of their calculated recurrence intervals.

The 500-, 100-, 50-, 25-, 10-, 5-, and 2-year peak flows for the Presumpscot River in Westbrook (site 10) were calculated using the guidelines (Bulletin 17B) of the Interagency Advisory Committee on Water Data (1982). The calculations involve fitting the log-Pearson Type III probability distribution to the observed annual peaks at a site. Bulletin 17B provides a procedure for incorporating historical peak flow information into the recurrence interval calculations. The historical peak flow information provided by S. D. Warren Co. (explained further in the Historical Perspective on Flood section) was used in the Bulletin 17B analysis.

Table 3. Recurrence intervals, exceedance probabilities, and peak flows for the Presumpscot River at the S.D. Warren Bridge in Westbrook, Maine (site 10)
[cubic feet per second abbreviated as ft^3/s]

Recurrence interval	Exceedance probability	Peak flow (ft^3/s)
2 years	0.500	5,310
5 years	0.200	7,830
10 years	0.100	9,890
25 years	0.040	13,000
50 years	0.020	15,700
100 years	0.010	18,900
200 years[a]	0.005	22,400
500 years[a]	0.002	28,000

[a] Recurrence intervals greater than 100 years have a large uncertainty associated with them.

Historical Peak Flows

Measured peak flows from the October 1996 flood are compared to the highest peak flows previously known, in table 2 (page 18). All previously known flows are from the USGS annual report series "Water Resources Data - Maine" with the exception of many flows on the Presumpscot River in Westbrook. The flows at the USGS gaging stations were calculated by standard USGS methods (Rantz and others, 1982).

Historical peak flows for the Presumpscot River at the Saccarappa Dam in Westbrook (which is about 1000 feet downstream from Site 6, see figure 9 for site locations) were supplied by S.D. Warren Co. (Tom Howard, written commun., 1996). The flows were calculated using weir computations. Some historical flows on the Presumpscot River are also available from the USGS at stream gaging stations in Westbrook (site 10, USGS station 01064118) and Falmouth (site 12, USGS station 01064140). The October 1996 peak flow for the Presumpscot River at I-95 in Falmouth (site 14) was determined by using a contracted opening indirect measurement (Matthai, 1967). The difference in drainage area between Saccarappa Dam and the USGS gaging station at Westbrook (site 10) is less than 2%, making the flows at the two sites comparable. Flows from the USGS Falmouth gaging station and from the I-95 bridge were adjusted by drainage area corrections (Morrill, 1975) to make them comparable to the flows at the USGS gaging station in Westbrook. All known significant historical peak flows on the Presumpscot River are listed in table 5. The October 1996 peak flow of 23,300 cubic feet per second was 68 percent larger than any other flow in the last 102 years.

Table 5. Significant historical peak flows on the Presumpscot River in Westbrook, Maine

Date	Flow (cubic feet per second)
April 15, 1895	13,000[a]
March 1, 1896	13,800[a]
February 13, 1900	11,300[a]
March 2, 1900	9,720[a]
May 17, 1916	12,400[a]
June 17, 1917	9,710[a]
March 12, 1936	11,200[a]
September 11, 1954	12,400[a]
March 14, 1977	11,200[b]
May 12, 1989	9,200[c]
August 20, 1991	13,900[c]
October 22, 1996	23,300[d]

[a] Flow computed by S.D. Warren Co. using weir equations at Saccarappa Dam in Westbrook.
[b] Flow computed by U.S. Geological Survey at streamflow gaging station in Falmouth and adjusted to Westbrook using a drainage area correction.
[c] Flow computed by U.S. Geological Survey at streamflow gaging station in Westbrook.
[d] Flow computed by U.S. Geological Survey by contracted opening measurement at I-95 bridge in Falmouth and adjusted to Westbrook using a drainage area correction.

Because no significant historical flows have been recorded for the Stroudwater River (site 28), Cascade Brook (site 33), or Capisic Brook (site 55), a site-specific historical perspective of the October 1996 flood at these sites is not possible. A general comparison, however, is possible. Crippen and Bue (1977) determined envelope curves for peak flow versus drainage area. These envelope curves were developed to provide a guide for estimating potential maximum flood flows. The curves were developed by analyzing thousands of sites with recorded flood flows, and using the sites with the most extreme flows to draw the envelope curves. The United States was divided into 17 different regions in the Crippen and Bue report. Maine is in region 1 (region 1 covers Maine, Vermont, most of New Hampshire, most of Connecticut, western Massachusetts, and a small part of Rhode Island) and this envelope curve is shown in figure 12. The points that were used to define the Crippen and Bue envelope curve for region 1 are listed in table 6. A general historical flood comparison is made by plotting the Presumpscot River (site 14), Stroudwater River (site the area severely affected by the October 1996 flood) are located on the Presumpscot River and are listed in table 7. The peak elevations for 1896, 1916, and 1954 were provided by S. D. Warren Co. (Tom Howard, written commun., 1996). The elevations for 1977, 1991, and 1996 are from USGS records. The peak elevations in table 7 were associated with the peak flows that are listed in table 5. The peak elevations from the October 1996 flood exceeded all previously recorded peak elevations on the lower Presumpscot River.

SUMMARY AND CONCLUSIONS

Flood flows are typically caused by a complex interrelationship of rainfall, rainfall intensity, antecedent drainage basin conditions, physical drainage basin characteristics and other factors. The cause of the southern Maine flood of October 1996, however, was not complex. Up to 19.19 inches of rain were recorded in southern Maine from October 20-22, 1996. The maximum 24-hour rainfall total of 13.32 inches at the Portland International Jetport exceeded the previous record (records have been kept since 1871) by 5.49 inches. Most of the rain from this storm fell in about 26 hours at a relatively constant rate (at the Jetport). None of the available data regarding hydrologic conditions prior to the October 20-22 storm were abnormally high or low. These facts and the strong relation (for this flood) between basin-average rainfall and the recurrence interval of peak flows indicate that the extreme rainfall totals from the October 1996 storm were the cause of the extreme peak streamflows experienced in southern Maine.

Table 7. Historical peak water-surface elevations on the Presumpscot River in Maine
[feet abbreviated as ft, na indicates not applicable, dash indicates not available]

Site	Location	Year					
		1896	1916	1954	1977	1991	1996
na	Upstream of Saccarappa Dam, Westbrook	-	73.8 ft	73.7 ft	-	-	76.8[a] ft
9	Upstream of Cumberland Mills Dam, Westbrook	-	46.0 ft	45.8 ft	-	-	51.6[b] ft
10	Upstream of S. D. Warren Bridge, Westbrook	-	-	-	-	38.9 ft	47.5 ft
11	Upstream of Route 302 Bridge, Westbrook	36.6 ft	34.1 ft	36.5 ft	-	-	43.8 ft
11	Downstream of Route 302 Bridge, Westbrook	-	33.9 ft	35.5 ft	-	-	-
12	Upstream of Blackstrap Rd. Bridge, Falmouth	35.8 ft	32.0 ft	33.5 ft	32.3 ft	-	41.0 ft
13	Upstream of Route 26/100 Bridge, Falmouth	35.1 ft	31.7 ft	31.8 ft	-	-	38.8 ft
15	Upstream of Smelt Hill Dam, Falmouth	22.5 ft	25.2 ft	23.4 ft	-	-	26.5 ft

[a] About 1000 feet upstream of Saccarappa Dam at Site 6.

[b] This peak water-surface elevation on the upstream side of Cumberland Mills Dam is the downstream elevation of the Cumberland Street bridge (see table 1).

Peak water-surface elevations from the October 1996 flood were determined at 74 locations. Peak flows were calculated at 13 sites and recurrence intervals were calculated at 12 of these 13 sites. Three of these flows had recurrence intervals of greater than 500 years. The peak flow on the Presumpscot River in Westbrook, Maine had a recurrence interval of 250 years. Peak-flow recurrence intervals greater than 100 years have a large amount of uncertainty associated with them. Based on historical data gathered for this report, the October 1996 peak flow on the Presumpscot River in Westbrook was 68 percent larger than any other flow at this location in the last 102 years. All known significant historical peak flows and peak elevations (in the area covered by this report) have been presented.

A strong positive relation was found between basin-average rainfalls and the recurrence intervals of peak flows from the October 1996 flood (in the area covered by this study). Based on this relation, it is probable that any drainage basin with an basin-average rainfall of more than 15 inches experienced a peak flow (at the drainage basin outlet) with a recurrence interval of greater than 100 years. Drainage basins with average rainfalls of less than 11 inches probably had peak flows with less than 100-year recurrence intervals. Those with less than 10 inches of rain likely had peak flows with less than 50-year recurrence intervals. Because of the strong relation between basin-average rainfall and peak-flow recurrence interval, peak-flow recurrence intervals for the October 1996 flood can be estimated for sites where peak flows were not determined (for basins in the area covered by this study with drainage areas greater than 1.5 square miles).

The October 1996 flood devastated parts of southern Maine. One person drowned after driving into floodwaters. More than 2,100 homes and businesses were damaged in the flood. Bridges, dams, or both were destroyed in Cape Elizabeth, Falmouth, Old Orchard Beach, Scarborough, Wells, South Berwick, Westbrook, and Windham. Clam flats from Kittery to Rockland were closed due to pollution.

Sebago Lake stored most of the rain that fell on the 441 square miles of the Presumpscot River drainage basin that is above the outlet of the lake. If this water had not been stored, peak flows and peak water-surface elevations on the lower Presumpscot River would have been even higher than they were in October 1996.

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Prepared in cooperation with the
Federal Emergency Management Agency

Flood of April 2007 in Southern Maine

Scientific Investigations Report 2009–5102

U.S. Department of the Interior
U.S. Geological Survey

Table 1. Peak water-surface elevations for April 2007 flood in York and Cumberland Counties, Maine.

[HWM, high-water mark; Northings and Eastings referenced to Maine State Plane West, in feet relative to North American Datum of 1983; elevations, in feet, referenced to North American Vertical Datum of 1988; Rt., route; DS, downstream; LE, left edge of water (when facing downstream); US, upstream; RE, right edge of water; Rd., Road; St., Street; Ln., Lane; RR, railroad; Br., bridge; E Br, East Branch; site number, location shown on figure 2]

Site number	HWM number	Easting (feet)	Northing (feet)	Elevation (feet)	Water body	Location and description
1	1	2777709.5	238615.3	426.10	Mousam River	Emery's Mills/Rt. 11/109, DS LE
1	2	2777635.6	238610.8	426.34	Mousam River	Emery's Mills/Rt. 11/109, US LE
2	3	2800776.0	213744.0	238.47	Mousam River	Rt. 4, US RE
2	4	2800876.8	213829.0	238.37	Mousam River	Rt. 4, US LE1
2	5	2800902.0	213868.4	238.51	Mousam River	Rt. 4, US LE2
2	6	2801016.8	213731.4	235.57	Mousam River	Rt. 4, DS LE
3	7	2822128.8	213453.7	172.52	Mousam River	Whichers Mill Rd., US RE1
3	8	2822133.1	213451.7	172.45	Mousam River	Whichers Mill Rd., US RE2
3	9	2822159.7	213445.0	172.52	Mousam River	Whichers Mill Rd., US RE3
4	10	2841465.4	208168.0	87.96	Mousam River	Mill St./Thompson St., DS LE1
4	11	2841479.2	208170.5	88.13	Mousam River	Mill St./Thompson St., DS LE2
4	12	2841250.4	208291.6	89.02	Mousam River	Mill St./Thompson St., US LE
5	13	2849520.4	204844.4	49.92	Mousam River	Intervale Rd., US LE 1
5	14	2849528.3	204854.4	49.86	Mousam River	Intervale Rd., US LE 2
5	15	2849590.0	204826.3	49.91	Mousam River	Intervale Rd., US LE 3
5	16	2849839.3	204784.5	49.64	Mousam River	Intervale Rd., US LE 4
5	17	2849844.4	204783.6	49.57	Mousam River	Intervale Rd., US LE 5
6	18	2851279.6	203281.8	47.58	Mousam River	Partridge Ln., US LE 1
6	19	2851294.2	203244.4	47.56	Mousam River	Partridge Ln., US LE 2
7	20	2852757.9	201312.0	39.30	Mousam River	Rt. 1, Top of Dam
7	21	2852662.8	201290.8	46.13	Mousam River	Rt. 1, US RE1
7	22	2852670.9	201314.2	46.12	Mousam River	Rt. 1, US RE2
8	23	2767009.5	177573.9	233.17	Little River	Long Swamp Rd., US LE
8	24	2766899.4	177558.6	231.07	Little River	Long Swamp Rd., DS LE
8	25	2766919.0	177650.5	230.73	Little River	Long Swamp Rd., DS RE
9	26	2762845.3	175526.1	197.37	Little River	Ridlon Rd., US RE
9	27	2762765.2	175465.2	196.42	Little River	Ridlon Rd., DS RE
10	28	2758042.6	172457.9	187.19	Little River	Hubbard Rd., DS RE1
10	29	2758042.6	172457.9	187.21	Little River	Hubbard Rd., DS RE2
10	30	2757953.1	172421.6	187.23	Little River	Hubbard Rd., DS RE3
10	31	2757947.1	172416.9	187.12	Little River	Hubbard Rd., DS RE4
11	32	2760177.4	182225.6	210.08	Keay Brook	Ridlon Rd., US RE
11	33	2760215.9	182014.4	204.31	Keay Brook	Ridlon Rd., DS LE1
11	34	2760190.7	182089.8	203.72	Keay Brook	Ridlon Rd., DS LE2
12	35	2831932.4	131286.3	18.31	Cape Neddick River	Rt. 1, DS LE
12	36	2831875.9	131198.6	21.50	Cape Neddick River	Rt. 1, US RE
13	37	2829182.4	157694.3	90.43	Ogunquit River	North Village Rd., US RE
13	38	2829337.6	157639.3	89.36	Ogunquit River	North Village Rd., DS LE
14	39	2837991.9	159806.6	32.51	Ogunquit River	Rt. 1, DS LE
14	40	2837843.9	159818.1	40.27	Ogunquit River	Rt. 1, US RE
15	41	2845139.6	182424.8	44.10	Blacksmith Brook	Rt. 1, US LE
15	43	2845258.8	182362.6	41.23	Blacksmith Brook	Rt. 1, DS LE

Table 1. Peak water-surface elevations for April 2007 flood in York and Cumberland Counties, Maine.—Continued

[HWM, high-water mark; Northings and Eastings referenced to Maine State Plane West, in feet relative to North American Datum of 1983; elevations, in feet, referenced to North American Vertical Datum of 1988; Rt., route; DS, downstream; LE, left edge of water (when facing downstream); US, upstream; RE, right edge of water; Rd., Road; St., Street; Ln., Lane; RR, railroad; Br., bridge; E Br, East Branch; site number, location shown on figure 2]

Site number	HWM number	Easting (feet)	Northing (feet)	Elevation (feet)	Water body	Location and description
16	44	2847370.8	186299.2	23.39	Merriland River	RR Br US from Rt. 1, DS RE1
16	45	2847407.4	186282.3	23.50	Merriland River	RR Br US from Rt. 1, DS RE2
16	46	2847265.3	186375.3	24.11	Merriland River	RR Br US from Rt. 1, US RE1
16	47	2847278.7	186372.1	24.09	Merriland River	RR Br US from Rt. 1, US RE2
17	48	2841932.6	199288.0	52.48	Branch Brook	Rt. 9a, US RE
18	49	2857289.8	264071.8	76.46	Stackpole Brook	Simpson Rd., US RE
18	50	2857251.1	264031.9	63.72	Stackpole Brook	Simpson Rd., DS RE
19	51	2835988.6	227721.0	143.46	Kennebunk River	Dam above Rt. 35, US RE1
19	52	2835991.3	227763.9	142.68	Kennebunk River	Dam above Rt. 35, US RE2
19	53	2835989.7	227731.8	142.77	Kennebunk River	Dam above Rt. 35, US RE3
20	54	2836244.3	227871.3	125.30	Kennebunk River	Rt. 35, DS LE
20	55	2836160.6	227768.7	133.13	Kennebunk River	Rt. 35, US RE1
21	56	2850061.2	218542.0	70.33	Kennebunk River	Downing Rd., US LE
21	57	2850095.2	218447.7	70.70	Kennebunk River	Downing Rd., DS LE1
22	58	2857176.4	207683.9	42.88	Kennebunk River	Rt. 1, DS RE
22	59	2857107.2	207786.7	44.92	Kennebunk River	Rt. 1, US RE
23	60	2792149.2	212320.3	229.40	Great Works River	Old Mill Rd., DS LE
23	61	2792138.8	212401.5	233.36	Great Works River	Old Mill Rd., US LE
24	62	2795265.4	203635.1	220.20	Great Works River	Sand Pond Rd., DS LE
24	63	2795216.1	203717.6	223.82	Great Works River	Sand Pond Rd., US LE
25	64	2799524.3	186930.9	188.11	Great Works River	Ford Quint Rd., US LE
25	65	2799481.7	186968.2	188.57	Great Works River	Ford Quint Rd., US RE
25	66	2799427.5	186933.4	187.00	Great Works River	Ford Quint Rd., DS RE
26	67	2797213.4	182553.0	178.36	Great Works River	Oak Woods Rd., US RE
26	68	2797185.4	182436.9	175.44	Great Works River	Oak Woods Rd., DS RE
27	69	2800512.2	176994.3	129.45	Great Works River	Rt. 4, DS LE
27	70	2800370.4	177057.4	131.58	Great Works River	Rt. 4, US LE
28	71	2800865.6	172981.7	125.14	Great Works River	Dam off Canal St. above Rt. 9, US LE
29	72	2803304.3	151303.7	94.55	Great Works River	Hoopers Sands Rd.-Emerys Br., DS RE
29	73	2803405.7	151236.1	97.30	Great Works River	Hoopers Sands Rd.-Emerys Br., US LE
30	74	2800913.5	171108.5	113.09	Great Works River	Madison St., DS LE1
30	75	2800786.8	171051.0	112.96	Great Works River	Madison St., DS LE2
31	76	2786140.1	140056.7	83.80	Great Works River	Rt. 236., DS LE
31	77	2786233.9	140094.7	84.97	Great Works River	Rt. 236., US LE
32	78	2784909.2	141101.7	82.38	Great Works River	Brattle St. Dam, US RE
32	79	2784848.3	141156.0	82.06	Great Works River	Brattle St. Dam, DS RE
33	80	2771946.6	257997.9	559.98	Pump Box Brook	Rt. 11, US LE
33	81	2771879.8	258044.1	555.50	Pump Box Brook	Rt. 11, DS LE1
33	82	2771871.0	258049.4	554.94	Pump Box Brook	Rt. 11, DS LE2
34	83	2762605.9	281713.5	472.30	Little Ossipee River	Rt. 11, DS LE
34	84	2762430.9	281752.0	479.64	Little Ossipee River	Rt. 11, US RE1
34	85	2762354.7	281750.4	485.95	Little Ossipee River	Rt. 11, US RE2

Table 1. Peak water-surface elevations for April 2007 flood in York and Cumberland Counties, Maine.—Continued

[HWM, high-water mark; Northings and Eastings referenced to Maine State Plane West, in feet relative to North American Datum of 1983; elevations, in feet, referenced to North American Vertical Datum of 1988; Rt., route; DS, downstream; LE, left edge of water (when facing downstream); US, upstream; RE, right edge of water; Rd., Road; St., Street; Ln., Lane; RR, railroad; Br., bridge; E Br, East Branch; site number, location shown on figure 2]

Site number	HWM number	Easting (feet)	Northing (feet)	Elevation (feet)	Water body	Location and description
35	86	2772624.7	297439.1	381.80	Little Ossipee River	Bridge St., DS RE1
35	87	2772646.8	297444.7	381.50	Little Ossipee River	Bridge St., DS RE2
35	88	2772570.5	297413.4	384.27	Little Ossipee River	Bridge St., US RE
36	89	2795457.3	299389.5	316.38	Little Ossipee River	Rt. 5, US LE
36	90	2795595.6	299339.5	316.38	Little Ossipee River	Rt. 5, DS LE
37	91	2819591.2	312426.4	284.17	Little Ossipee River	Sand Pond Rd., US RE
37	92	2819371.0	312388.0	280.91	Little Ossipee River	Sand Pond Rd., DS LE
38	93	2913214.7	395360.5	181.85	Collyer Brook	Rt. 202, DS RE1
38	94	2913229.0	395356.0	181.24	Collyer Brook	Rt. 202, DS RE2
38	95	2913186.7	395401.5	183.25	Collyer Brook	Rt. 202, DS RE3
38	96	2913228.9	395440.4	181.34	Collyer Brook	Rt. 202, DS LE
38	97	2913113.4	395508.0	185.56	Collyer Brook	Rt. 202, US LE
38	98	2913088.0	395458.7	186.06	Collyer Brook	Rt. 202, US RE
39	99	2949539.8	351984.5	16.67	Royal River	Yarmouth, US RE1
39	100	2949542.0	351983.7	16.55	Royal River	Yarmouth, US RE2
39	101	2949537.0	351979.6	16.76	Royal River	Yarmouth, US RE3
40	102	2930171.7	342925.9	42.54	E Br Piscataquis River	Rt. 9, DS LE
40	103	2930070.9	342859.3	42.80	E Br Piscataquis River	Rt. 9, DS RE
40	104	2930215.8	342942.0	43.59	E Br Piscataquis River	Rt. 9, US LE
40	105	2930194.9	342992.4	43.72	E Br Piscataquis River	Rt. 9, US RE
41	106	2919250.9	329761.3	34.34	Piscataquis River	Leighton Rd., US LE
41	107	2919201.3	329655.5	33.23	Piscataquis River	Leighton Rd., DS RE
42	108	2914582.0	300009.1	23.29	Stroudwater River	Dam above Westbrook St., US RE
43	109	2913643.6	300350.7	25.35	Stroudwater River	Congress St., US RE
43	110	2913828.8	300220.5	24.64	Stroudwater River	Congress St., DS RE
44	111	2908651.6	294653.3	39.27	Long Creek	Maine Mall Rd., DS RE
44	112	2908541.0	294804.5	42.30	Long Creek	Maine Mall Rd., US RE
45	113	2932033.0	275801.5	7.41	Spurwink River	Spurwink Rd., US LE
45	114	2932022.2	275832.9	7.32	Spurwink River	Spurwink Rd., US RE
45	115	2932022.4	275839.2	7.33	Spurwink River	Spurwink Rd., US RE
45	116	2931920.5	275674.7	6.50	Spurwink River	Spurwink Rd., DS LE
46	117	2883697.0	287367.7	38.79	Nonesuch River	Mitchell Hill Rd., DS LE
47	118	2906547.5	284595.5	17.21	Nonesuch River	Rt. 114, DS LE
48	119	2910487.3	272125.4	7.41	Nonesuch River	Blackpoint Rd., DS LE
48	120	2910536.7	272146.1	7.56	Nonesuch River	Blackpoint Rd., US LE1
48	121	2910506.4	272158.7	7.40	Nonesuch River	Blackpoint Rd., US LE2
49	122	2846112.1	424970.0	289.98	Crooked River	Edes Falls Rd., US LE1
49	123	2846103.8	424946.2	289.90	Crooked River	Edes Falls Rd., US LE2
49	124	2846110.5	424907.2	289.94	Crooked River	Edes Falls Rd., US LE3

Table 1. Peak water-surface elevations for April 2007 flood in York and Cumberland Counties, Maine.—Continued

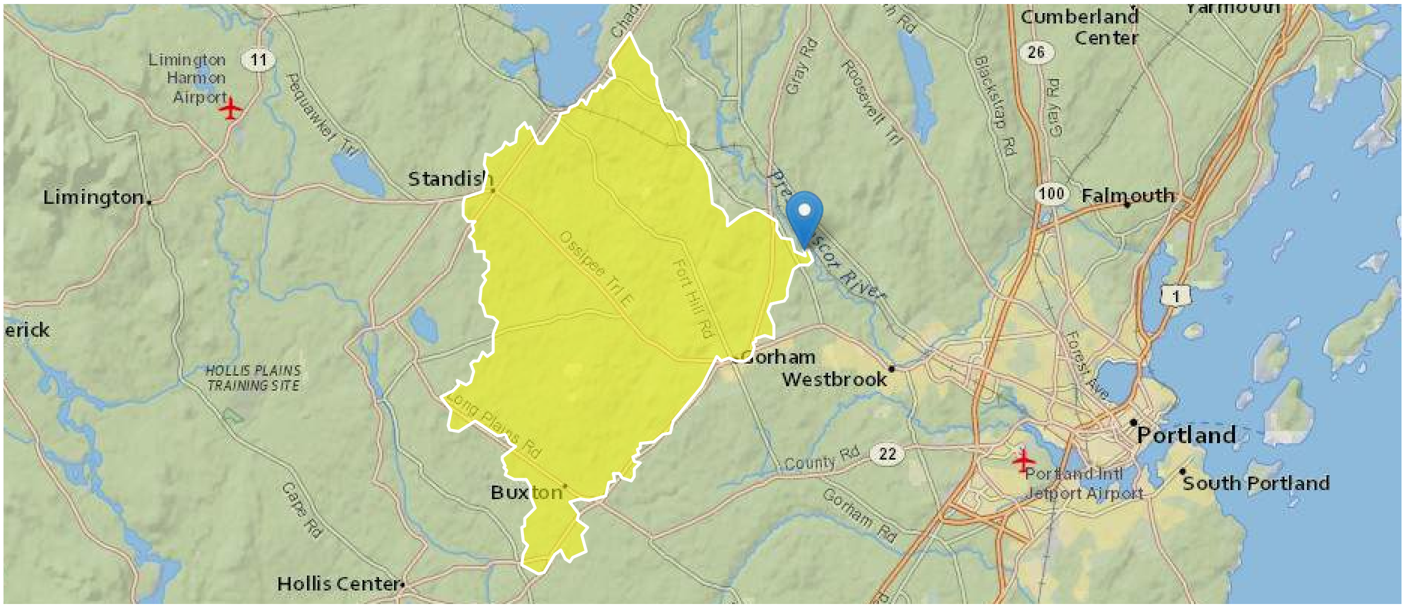
[HWM, high-water mark; Northings and Eastings referenced to Maine State Plane West, in feet relative to North American Datum of 1983; elevations, in feet, referenced to North American Vertical Datum of 1988; Rt., route; DS, downstream; LE, left edge of water (when facing downstream); US, upstream; RE, right edge of water; Rd., Road; St., Street; Ln., Lane; RR, railroad; Br., bridge; E Br, East Branch; site number, location shown on figure 2]

Site number	HWM number	Easting (feet)	Northing (feet)	Elevation (feet)	Water body	Location and description
49	125	2846142.3	424787.9	288.95	Crooked River	Edes Falls Rd., DS LE1
49	126	2846177.8	424766.6	288.67	Crooked River	Edes Falls Rd., DS LE2
49	127	2846002.6	424735.0	288.90	Crooked River	Edes Falls Rd., DS RE
50	128	2848198.8	417923.6	283.08	Crooked River	Rt. 11, US RE
50	129	2848331.4	418021.9	283.48	Crooked River	Rt. 11, US LE
50	130	2848357.8	417940.4	283.23	Crooked River	Rt. 11, DS LE
50	131	2848318.2	417778.9	283.02	Crooked River	Rt. 11, DS RE1
50	132	2848283.9	417710.7	282.73	Crooked River	Rt. 11, DS RE2
51	133	2786002.3	355386.1	286.81	Ossipee River	River Rd., DS LE1
51	134	2785999.7	355335.2	286.58	Ossipee River	River Rd., DS LE2
51	135	2785934.6	355420.3	285.77	Ossipee River	River Rd., DS LE3
51	136	2785904.3	355596.0	287.21	Ossipee River	River Rd., US LE
51	137	2785786.6	355208.8	286.91	Ossipee River	River Rd., US RE
51	138	2785907.0	355153.1	286.74	Ossipee River	River Rd., DS RE
52	139	2794615.6	348347.8	327.25	Pease Brook	Rt. 25, US RE1
52	140	2794552.7	348369.9	327.47	Pease Brook	Rt. 25, US LE1
52	141	2794537.0	348367.4	327.51	Pease Brook	Rt. 25, US LE2
52	142	2794540.8	348330.6	327.36	Pease Brook	Rt. 25, US LE3
52	143	2794667.5	348432.3	321.68	Pease Brook	Rt. 25, DS LE2
53	144	2884574.6	323620.6	84.00	Little River	Rt. 202, US RE
53	145	2884763.8	323526.2	83.66	Little River	Rt. 202, DS RE
54	146	2887332.4	320867.4	81.80	Little River	Rt. 237, US RE
54	147	2887356.7	320867.1	81.69	Little River	Rt. 237, US RE
55	148	2885556.2	347553.1	163.37	Pleasant River	Pope Rd., US RE1
55	149	2885606.8	347569.8	163.37	Pleasant River	Pope Rd., US RE2
55	150	2885479.1	347448.3	160.84	Pleasant River	Pope Rd., DS LE
56	151	2834138.2	124573.7	9.28	Briley Stream	Bay St., HWM1
56	152	2834111.0	124629.7	9.35	Briley Stream	Bay St., HWM2
57	153	2889894.8	257501.8	40.30	Mill Brook	Rt. 98, DS LE
57	154	2889793.3	257481.9	43.90	Mill Brook	Rt. 98, US LE
58	155	2805301.1	261125.7	259.82	Carl Branch Brook	Straw Mill Rd., DS RE
58	156	2805363.8	261235.0	262.66	Carl Branch Brook	Straw Mill Rd., US RE
59	157	2923162	376291	247.01	Unnamed Brook	Gray crest-stage gage
60	158	2940962	339060.4	69.07	Chenery Brook	Cumberland crest-stage gage
61	159	2827942.4	372928.2	282.5	Stony Brook	Rt. 11, at the streamflow-gaging station
62	160	2790435.1	355868.6	274.1	Saco River	Rt. 117, at the streamflow-gaging station
63	161	2904990.5	311177.2	38.03[1]	Presumpscot River	Westbrook, at the streamflow-gaging station

[1]27.73 feet NAVD 88 is flood stage at this location.

Little River

Region ID: ME
Workspace ID: ME20180810145506582000
Clicked Point (Latitude, Longitude): 43.71572, -70.40983
Time: 2018-08-10 10:55:22 -0400



Basin Characteristics

Parameter Code	Parameter Description	Value	Unit
DRNAREA	Area that drains to a point on a stream	49.8	square miles
STORNWI	Percentage of strorage (combined water bodies and wetlands) from the Nationa Wetlands Inventory	11.76	percent

Peak-Flow Statistics Parameters [Statewide Peak Flow Full GT 12sqmi WRI 99 4008]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	49.8	square miles	0.93	1653
STORNWI	Percentage of Storage from NWI	11.76	percent	0.7	26.7

Peak-Flow Statistics Flow Report [Statewide Peak Flow Full GT 12sqmi WRI 99 4008]

PII: Prediction Interval-Lower, Plu: Prediction Interval-Upper, SEp: Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unit	PII	Plu	SE	SEp	Equiv. Yrs.
2 Year Peak Flood	1140	ft^3/s	637	2040	35.1	35.1	1.8
5 Year Peak Flood	1700	ft^3/s	942	3070	36.1	36.1	2.5
10 Year Peak Flood	2110	ft^3/s	1150	3860	36.8	36.8	3.2
25 Year Peak Flood	2640	ft^3/s	1400	4950	38.6	38.6	4.1
50 Year Peak Flood	3050	ft^3/s	1590	5840	39.9	39.9	4.8

Statistic	Value	Unit	PII	PIu	SE	SEp	Equiv. Yrs.
100 Year Peak Flood	3480	ft^3/s	1780	6820	41.2	41.2	5.4
500 Year Peak Flood	4550	ft^3/s	2200	9410	44.9	44.9	6.4

Peak-Flow Statistics Citations

Hodgkins, G. A.,1999, Estimating the Magnitude of Peak Flows for Streams in Maine for Selected Recurrence Intervals: U.S. Geological Survey Water-Resources Investigations Report 99-4008, 45 p. (http://me.water.usgs.gov/99-4008.pdf)

Application Version: 4.2.1

Hoyle, Tanner & Associates, Inc.
150 Dow Street
Manchester, NH 03101

Ret Pd T (yr)	Peak Flow Estimate Lower	Q_T (ft^3/s)	Upper
2	637.00	1140.0	2040.0
5	942.00	1700.0	3070.0
10	1150.00	2110.0	3860.0
25	1400.00	2640.0	4950.0
50	1590.00	3050.0	5840.0
100	1780.00	3480.0	6820.0
500	2200.00	4500.0	9410.0

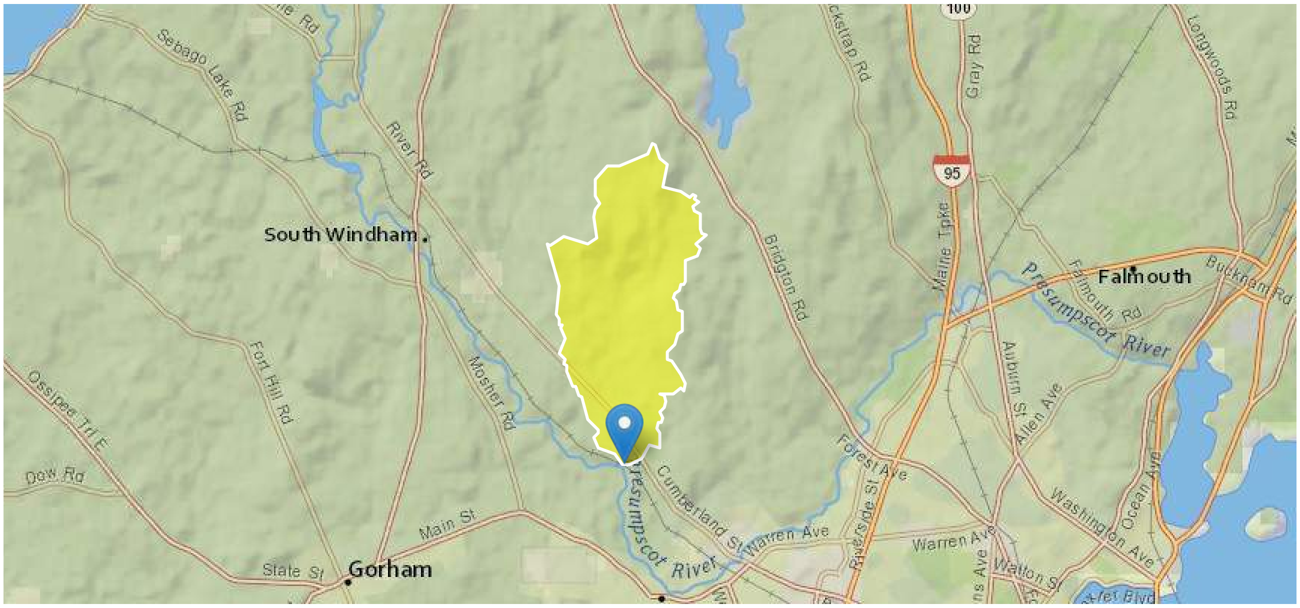
μ (log) 1.76
s (log) 2.307E-01

Flood Frequency Curve

Ret Pd T	Cum Prob F	Q_T (m3/s usgs)	Log-Normal Distn norm std var z	log (Q m3/s)	Q (m^3/s)	Q (ft^3/s)
1.001	0.001		-3.091	1.05	11.19	395.04
1.002	0.002		-2.879	1.10	12.52	442.07
1.010	0.010		-2.330	1.22	16.75	591.67
1.050	0.048		-1.668	1.38	23.81	840.87
1.100	0.091		-1.335	1.45	28.42	1003.70
1.250	0.200		-0.842	1.57	36.94	1304.57
1.500	0.333		-0.431	1.66	45.95	1622.78
1.750	0.429		-0.180	1.72	52.50	1853.96
2	0.500	57.77	0.000	1.76	57.77	2040.00
5	0.800	86.94	0.842	1.96	90.33	3190.02
6.300	0.841		1.000	1.99	98.25	3469.45
10	0.900	109.31	1.282	2.06	114.12	4029.83
20	0.950		1.645	2.14	138.41	4887.66
25	0.960	140.17	1.751	2.17	146.41	5170.31
50	0.980	165.38	2.054	2.24	171.99	6073.42
100	0.990	193.13	2.326	2.30	198.78	7019.76
200	0.995		2.576	2.36	226.95	8014.54
500	0.998	266.47	2.878	2.43	266.49	9410.81
1000	0.999		3.090	2.47	298.27	10533.00

Inkhorn Brook

Region ID: ME
Workspace ID: ME20180824171827165000
Clicked Point (Latitude, Longitude): 43.69878, -70.37829
Time: 2018-08-24 13:18:42 -0400



Basin Characteristics

Parameter Code	Parameter Description	Value	Unit
DRNAREA	Area that drains to a point on a stream	3.9	square miles
STORNWI	Percentage of strorage (combined water bodies and wetlands) from the Nationa Wetlands Inventory	6.49	percent

Peak-Flow Statistics Parameters [Statewide Peak Flow DA LT 12sqmi 2015 5049]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	3.9	square miles	0.31	12
STORNWI	Percentage of Storage from NWI	6.49	percent	0	22.2

Peak-Flow Statistics Flow Report [Statewide Peak Flow DA LT 12sqmi 2015 5049]

PII: Prediction Interval-Lower, Plu: Prediction Interval-Upper, SEp: Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unit	SEp
1.01 Year Peak Flood	44.3	ft^3/s	38
2 Year Peak Flood	153	ft^3/s	34
5 Year Peak Flood	243	ft^3/s	35
10 Year Peak Flood	308	ft^3/s	37
25 Year Peak Flood	403	ft^3/s	39

Statistic	Value	Unit	SEp
50 Year Peak Flood	474	ft^3/s	41
100 Year Peak Flood	555	ft^3/s	42
250 Year Peak Flood	631	ft^3/s	44
500 Year Peak Flood	752	ft^3/s	47

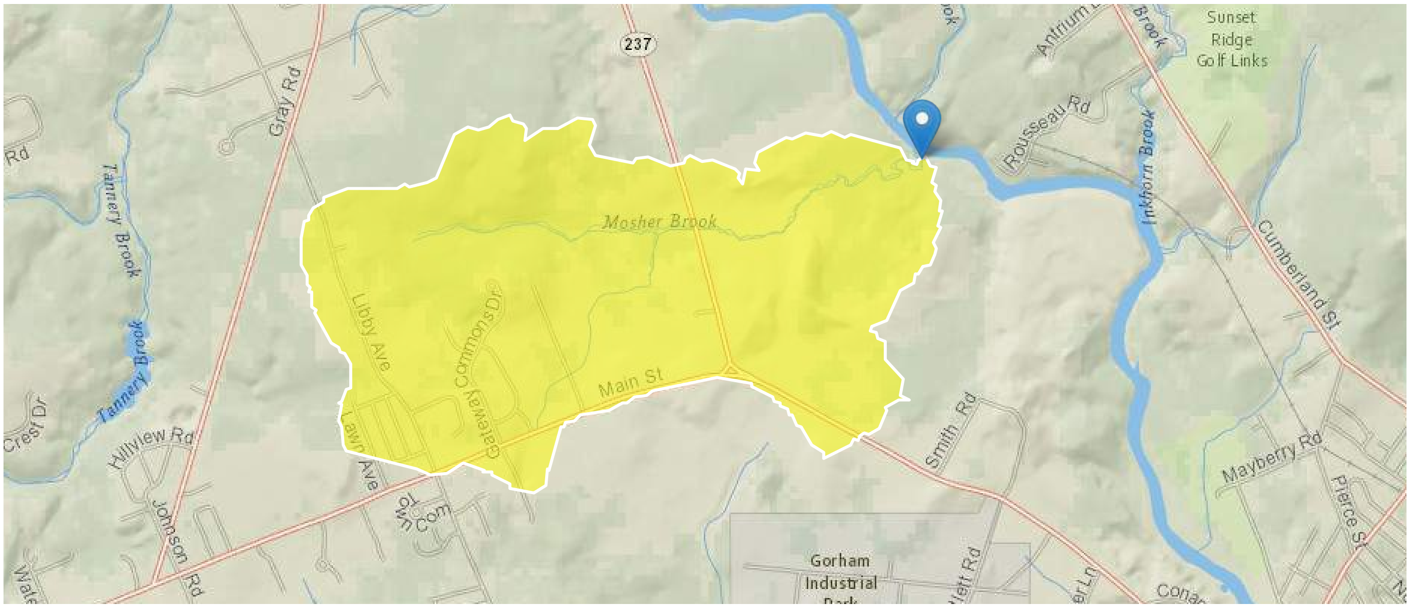
Peak-Flow Statistics Citations

Lombard, P.J., and Hodgkins, G.A.,2015, Peak flow regression equations for small, ungaged streams in Maine— Comparing map-based to field-based variables: U.S. Geological Survey Scientific Investigations Report 2015–5049, 12 p. (http://dx.doi.org/10.3133/sir20155049)

Application Version: 4.2.1

Mosher Brook

Region ID: ME
Workspace ID: ME20180826010609439000
Clicked Point (Latitude, Longitude): 43.69981, -70.39099
Time: 2018-08-25 21:06:24 -0400



Basin Characteristics

Parameter Code	Parameter Description	Value	Unit
DRNAREA	Area that drains to a point on a stream	1.3	square miles
STORNWI	Percentage of strorage (combined water bodies and wetlands) from the Nationa Wetlands Inventory	4.34	percent

Peak-Flow Statistics Parameters [Statewide Peak Flow DA LT 12sqmi 2015 5049]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	1.3	square miles	0.31	12
STORNWI	Percentage of Storage from NWI	4.34	percent	0	22.2

Peak-Flow Statistics Flow Report [Statewide Peak Flow DA LT 12sqmi 2015 5049]

PII: Prediction Interval-Lower, Plu: Prediction Interval-Upper, SEp: Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unit	SEp
1.01 Year Peak Flood	20.2	ft^3/s	38
2 Year Peak Flood	68.1	ft^3/s	34
5 Year Peak Flood	108	ft^3/s	35
10 Year Peak Flood	137	ft^3/s	37
25 Year Peak Flood	179	ft^3/s	39

Statistic	Value	Unit	SEp
50 Year Peak Flood	211	ft^3/s	41
100 Year Peak Flood	247	ft^3/s	42
250 Year Peak Flood	282	ft^3/s	44
500 Year Peak Flood	336	ft^3/s	47

Peak-Flow Statistics Citations

Lombard, P.J., and Hodgkins, G.A.,2015, Peak flow regression equations for small, ungaged streams in Maine— Comparing map-based to field-based variables: U.S. Geological Survey Scientific Investigations Report 2015–5049, 12 p. (http://dx.doi.org/10.3133/sir20155049)

Application Version: 4.2.1

DeGraff, Jeffrey A.

From: DeGraff, Jeffrey A.
Sent: Tuesday, August 14, 2018 3:54 PM
To: 923403.00 MaineDOT GCA Bridge Design
Subject: FW: [EXT] Saccarappa Dam information
Attachments: Saccarappa, Double Denil, Existing Site Plan SD-1, F, BG, 2-15-18.pdf

Jeffrey A. De Graff, Jr., PE
Structural & Hydraulic Engineer
Licensed in VT
Hoyle, Tanner & Associates, Inc.
(802) 860-1331 ext. 315

From: Stemm, Barry <Barry.Stemm@sappi.com>
Sent: Monday, August 13, 2018 11:38 AM
To: DeGraff, Jeffrey A. <jdegraff@hoyletanner.com>
Subject: RE: [EXT] Saccarappa Dam information

Jeffery,

We do not have a stage/discharge curve for the site that we can give you. However the information on this attached site plan should provide you with all the information needed to develop a curve. The elevation of the top of the spillways at Saccarappa is 70 ft. +/- 0.2 ft. based on the survey datum listed (NGVD 29). We recommend that you use only the spillway lengths from this drawing to develop the curve. The overflow spillway just upstream of the powerhouse is downstream of the headgates in the forebay channel and those gates are often closed during major flood events.

Best regards,
Barry

From: DeGraff, Jeffrey A. [mailto:jdegraff@hoyletanner.com]
Sent: Wednesday, August 08, 2018 10:45 AM
To: Stemm, Barry
Subject: RE: [EXT] Saccarappa Dam information

Barry,

Thank you.

Jeffrey A. De Graff, Jr., PE
Structural & Hydraulic Engineer
Licensed in VT
Hoyle, Tanner & Associates, Inc.
(802) 860-1331 ext. 315

From: Stemm, Barry <Barry.Stemm@sappi.com>
Sent: Wednesday, August 08, 2018 10:15 AM
To: DeGraff, Jeffrey A. <jdegraff@hoyletanner.com>
Subject: RE: [EXT] Saccarappa Dam information

Jeffery,

I will review if this information is something we are able to share and get back with you.

Barry Stemm
Engineering Manager

sappi
Inspired by life

Barry Stemm
Engineering Dept.
Sappi North America
89 Cumberland Street
PO Box 5000 | Westbrook, ME | 04092
Tel +1 207 856 4584 **| Mobile** +1 207 807 3974 **| Fax** +1 207 856 4456
Barry.Stemm@sappi.com | www.sappi.com

Paper is a sustainable and renewable source. Please recycle all printed documents. Paper informs, inspires and protects.

From: DeGraff, Jeffrey A. [mailto:jdegraff@hoyletanner.com]
Sent: Wednesday, August 08, 2018 9:51 AM
To: Stemm, Barry
Subject: [EXT] Saccarappa Dam information
Importance: High

Good morning Barry,

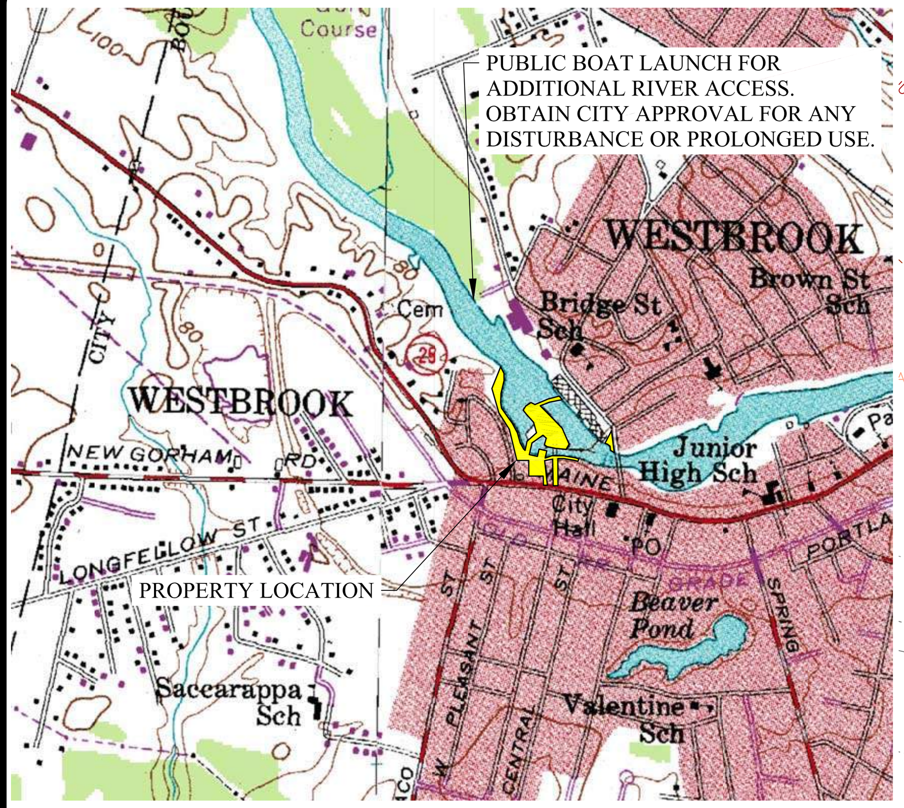
I have attached an email correspondence with Brad Goulet for reference. That said, I will be sizing a new bridge found at Mosher Road (237) over the Little River in Gorham, ME. I do understand that the Saccarappa Dam may be removed in the near future and am wondering if you could provide me with previous hydraulic studies for the removal? Or could send me contact information for the design engineer?

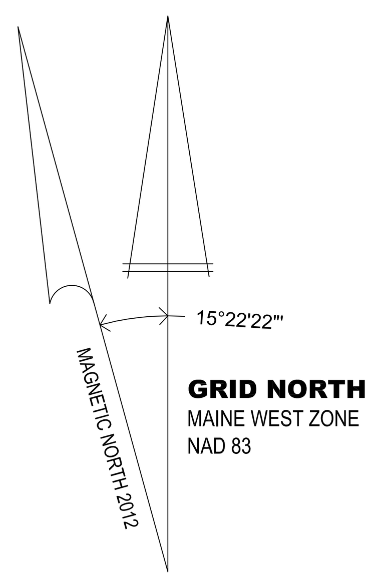
Any help would be beneficial.

Thank you and have a great day,

Jeffrey A. De Graff, Jr., PE
Structural & Hydraulic Engineer
Hoyle, Tanner & Associates, Inc.

Responsive. Consistent. Competent.™





GRID NORTH
MAINE WEST ZONE
NAD 83

SURVEY NOTES:

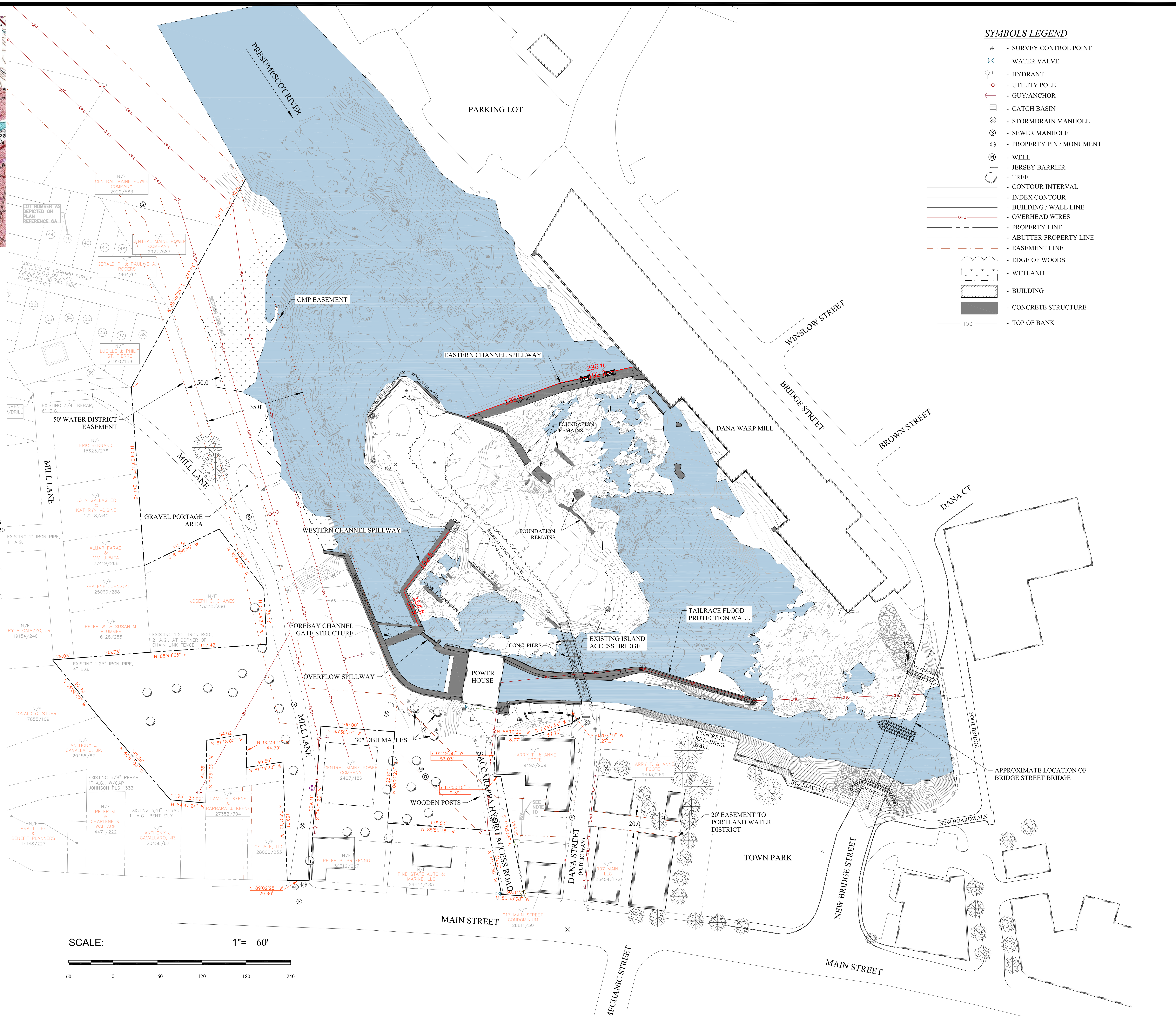
(1) ELEVATIONS SHOWN ARE NGVD 29 BASED UPON NGS DISK R121 WITH PUBLISHED ELEVATION OF 54.14'.

(2) CONTOURS SHOWN ARE 1 FOOT INTERVALS.

(3) DATA BASED ON FIELD SURVEYS BY PLISGA & DAY LAND SURVEYORS 26 SEPTEMBER, 2011, 14 NOVEMBER, 2011,13 DECEMBER, 2011, 19 MAY, 2013, 20 JUNE, 2013, AND 4 DECEMBER, 2013 USING A TRIMBLE S6 ROBOTIC TOTAL STATION AND A TDS RANGER DATA COLLECTOR; AND BY HARRY R. FELDMAN, INC 13 DECEMBER, 2011 USING A DIGITAL SCANNER.

(4) COORDINATES ARE ORIENTED TO STATE PLANE, MAINE WEST, ZONE 1802, WITH GROUND DISTANCES.

(5) BOUNDARIES, UNDERGROUND AND OVERHEAD UTILITIES SHOWN HEREON ARE BASED ON SURVEY PERFORMED BY SEBAGO TECHNICS, INC IN OCTOBER AND NOVEMBER 2013 AND SHOWN ON SITE PLAN DATED JUNE 9, 2014.



SYMBOLS LEGEND

- SURVEY CONTROL POINT
- WATER VALVE
- HYDRANT
- UTILITY POLE
- GUY/ANCHOR
- CATCH BASIN
- STORMDRAIN MANHOLE
- SEWER MANHOLE
- PROPERTY PIN / MONUMENT
- WELL
- JERSEY BARRIER
- TREE
- CONTOUR INTERVAL
- INDEX CONTOUR
- BUILDING / WALL LINE
- OVERHEAD WIRES
- PROPERTY LINE
- ABUTTER PROPERTY LINE
- EASEMENT LINE
- EDGE OF WOODS
- WETLAND
- BUILDING
- CONCRETE STRUCTURE
- TOP OF BANK

SCALE: 1"= 60'

60 0 60 120 180 240

DO NOT USE FOR CONSTRUCTION
ISSUED FOR BIDDING

No.	Revision Description	Drwn	Chk'd	Date
-	ISSUED FOR BIDDING	BPG	KJB	10/9/17

Drwn By: BPG
Desg By: -
Chkd By: KJB / WBB
Aprvd By: WBB
Date: 10/9/17

ACHERON ENGINEERING SERVICES
Engineering, Environmental & Geologic Consultants
www.AcheronEngineering.com
147 Main St. Newport, ME. 04953 (207)-368-5700
24466 Powell Rd. Brooksville, FL 34602 (352)-796-6236
Acheron International, Inc.

Saccarappa Falls Fish Passage Project
Two Channel with Double Denil
Existing Site Plan
Sappi / Warren Release Papers
Saccarappa Falls
Westbrook, Maine

Job Number: 49412

Drawing No: SD-1

Sheet 2 of 38

Hoyle, Tanner & Associates, Inc.
150 Dow Street
Manchester, NH 03101

Input

Weir Eq. (Broad)	Q = CLH^n
Weir Length	391 ft
Overflow Width	142 ft
Crest Elevation	70 ft (NGVD29)
Structure Varient	1.5

Legend

Q = Volumetric flow rate
C = Constant for the specific weir structure
L = Width of the weir
H = Height of water head upstream in relation to the weir's crest
n = structure varient (typical for horizontal weir)

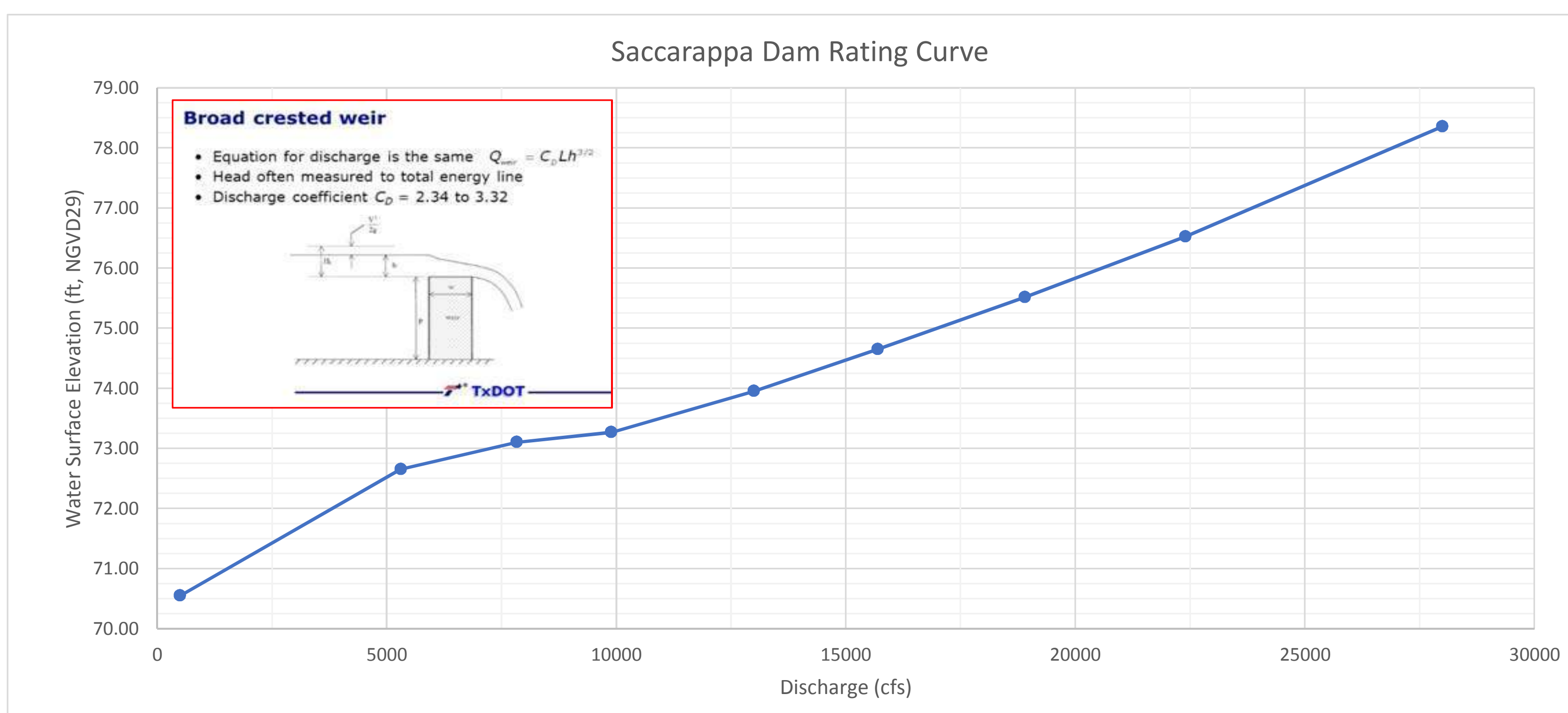
Calibrate Weir Coefficients to Match USGS 1996 Report Water Surface Elevations

Event	Q (cfs)	HW Elev. (ft)	HW Depth (ft)	Weir Coefficient, C	Qcheck	Reference	Notes
1916	12400	73.8	3.8	3.14	12400	Table 5, 7	During high flows, power house is not running. Based of contours, flow will be directed to overflow channel.
1996	23300	76.8	6.8	2.47	23300	Table 5, 7	During high flows, power house is not running. Based of contours, flow will be directed to overflow channel.

Develop Rating Curve for Saccarappa Dam

Storm Event	Q (cfs)	Wier Coef, C	HW Depth (ft)	HW Elev. (ft, NGVD29)	HW Elev. (ft, NAVD88)	Notes
Low Flow	500	3.141	0.55	70.55	69.86	Arbitrary low flow value
2yr	5310	3.141	2.65	72.65	71.96	Below Overflow Elevation of 72.00 +/-
5yr	7830	3.141	3.10	73.10	72.41	Assume 65-ft of overflow is accessed at western spillway
10yr	9890	3.141	3.27	73.27	72.58	Assume entire overlow channel is accessed
25yr	13000	3.103	3.95	73.95	73.26	Assume entire overlow channel is accessed
50yr	15700	2.936	4.65	74.65	73.96	Assume entire overlow channel is accessed
100yr	18900	2.738	5.51	75.51	74.82	Assume entire overlow channel is accessed
200yr	22400	2.521	6.53	76.53	75.83	Assume entire overlow channel is accessed
500yr	28000	2.174	8.36	78.36	77.67	Assume entire overlow channel is accessed
4/16/2007	11825	3.141	3.68	73.68	72.99	Flow determined using DS USGS Gage Stage Discharge Graph

Notes: Flows for storm events are from Table 3 of the 1996 flood report.
Assume dam efficiency is maintained below 12,400 cfs and decreases linearly above 12,400 cfs.



Used to determine 04/2007 Flow over Sacc Dam

National Weather Service Historical Crests

Historic Crests

(1) 34.10 ft on 10/22/1996
(2) 25.48 ft on 08/20/1991
(3) 25.30 ft on 04/16/2007
(4) 23.55 ft on 02/26/2010
(5) 22.26 ft on 05/12/1989
(6) 21.25 ft on 06/03/2012
(7) 20.88 ft on 10/10/1998
(8) 20.83 ft on 04/01/1987
(9) 19.18 ft on 06/14/1998
(10) 19.11 ft on 01/27/1986
(11) 18.55 ft on 09/07/2008
(12) 17.17 ft on 01/11/2016
(13) 17.12 ft on 06/17/1998
(14) 17.06 ft on 04/23/1991
(15) 16.95 ft on 09/30/2015
(16) 16.06 ft on 04/11/1993
(17) 16.00 ft on 04/13/1993
(18) 15.80 ft on 06/27/1998
(19) 15.70 ft on 12/13/2010
(20) 15.57 ft on 03/24/2010
(21) 15.52 ft on 04/02/2004
(22) 15.07 ft on 12/25/1994

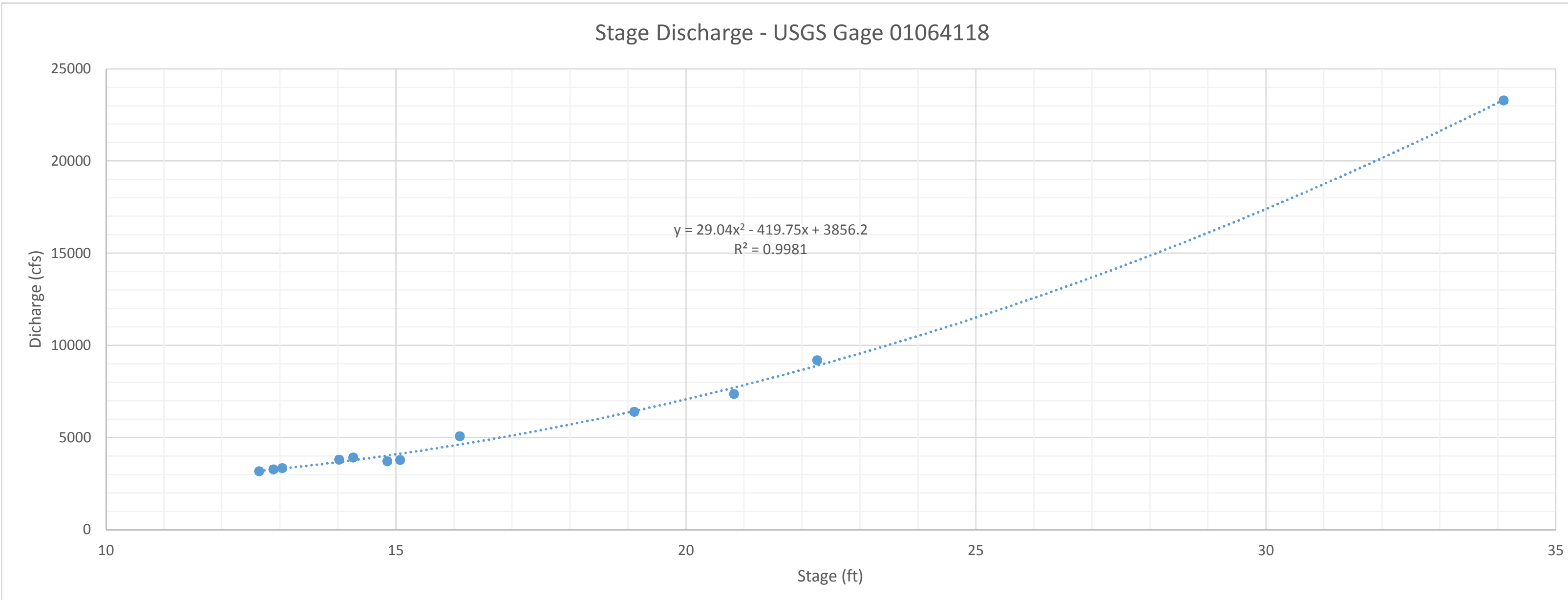
https://water.weather.gov/ahps2/hydrograph.php?wfo=gyx&gage=wbkm1
https://water.weather.gov/ahps2/crests.php?wfo=gyx&gage=wbkm1&crest_type=historic

USGS Gage 1064118 Historical Peaks

USGS Gage		Date	Flow	Stage Ht	Calc'd Discharge
USGS	1064118	3/12/1985	3920	14.26	3776
USGS	1064118	1/27/1986	6400	19.11	6440
USGS	1064118	4/1/1987	7360	20.83	7713
USGS	1064118	4/29/1988	3810	14.02	3679
USGS	1064118	5/12/1989	9200	22.26	8902
USGS	1064118	4/4/1990	3350	13.04	3321
USGS	1064118	3/11/1992	3280	12.89	3271
USGS	1064118	4/11/1993	5080	16.1	4626
USGS	1064118	12/22/1993	3720	14.85	4027
USGS	1064118	12/25/1994	3790	15.07	4126
USGS	1064118	10/22/1996	23300	34.1	23311
USGS	1064118	4/7/2017	3170	12.64	3190
National Weather Service Gage Height (4-16-07)				25.3	11825

https://nwis.waterdata.usgs.gov/me/nwis/peak/?site_no=01064118&agency_cd=USGS

Hoyle, Tanner Project No. 923403.08	Sheet:	2	of:	2
Little River Bridge	Calc By:	JAD	Date:	10/2018
Route 237 over Little River	Check By:	KMH	Date:	10/2018
Gorham, ME	Rev By:		Date:	
Determine 4/16/2007 flow @ Saccarappa Dam	Rev Check By:		Date:	



FULL APPLICATION

Mallison Falls Hydroelectric Facility

Certification Application to the Low Impact Hydropower Institute

FERC Project No. 2932

Prepared by
Peter Drown, President
Cleantech Analytics LLC
On Behalf of
S.D. Warren Company d/b/a
Sappi North America

September 1, 2017

	Dates, purpose, and type of any recent operational changes	N/A
	Plans, authorization, and regulatory activities for any facility upgrades	N/A
Characteristics of Dam, Diversion, or Conduit	Date of construction	1900
	Dam height	14 ft.
	Spillway elevation and hydraulic capacity	• Elevation: 90.5 ft. MSL • Hydraulic Capacity:
	Tailwater elevation	71.7 ft. MSL
	Length and type of all penstocks and water conveyance structures between reservoir and powerhouse	Intake Canal: • 675 ft. long • 41 ft. wide • 6 ft. deep
	Dates and types of major, generation-related infrastructure improvements	2016 Replace Canal Gates 2016 Repairs to Tailrace
	Designated facility purposes (e.g., power, navigation, flood control, water supply, etc.)	Electricity generation
	Water source	Presumpscot River
	Water discharge location or facility	Discharges into the Presumpscot River at river mile 16.4
Characteristics of Reservoir and Watershed	Gross volume and surface area at full pool	• Volume: n/a (run of river with limited ponding) • Surface Area: 8 acres
	Maximum water surface elevation (ft. MSL)	90.5 ft. MSL
	Maximum and minimum volume and water surface elevations for designated power pool, if available	N/A
	Upstream dam(s) by name, ownership, FERC number (if applicable), and river mile	**Name** / **FERC No.** / **Owner** / **RM** Eel Weir / P-2984 / S.D. Warren / 25 North Gorham[2] / P-2519 / Brookfield / 23.6 Dundee / P-2942 / S.D. Warren / 21.9 Gambo / P-2931 / S.D. Warren / 18.6 Little Falls / P-2941 / S.D. Warren / 16.9
	Downstream dam(s) by name,	**Name** / **FERC** / **Owner** / **RM**

[2] North Gorham received LIHI Certification on April 27, 2016 (Certificate #129).

Hydraulic Analysis Software

The software package that was used to develop the SRH-2D (Sediment and River Hydraulics, Two-Dimensional) model for the existing and proposed crossings is Aquaveo's Surface-water Modeling System (SMS) Version 12.3. The program allows the user to develop a two-dimensional (2D) hydraulic, sediment and temperature model that incorporates the Finite Volume method in conjunction with implicit first- and second-order numerical schemes to approximate a solution for the 2D depth averaged Saint Venant equations.

Field Data

Field measurements were not taken for this project but USGS Water-Resources Investigation Report 97-4189, titled "Flood of October 1996 in Southern Maine" was utilized for calibration purposes. This report is attached to this document.

Hydrology

Peak flows for the Q2, Q10, Q25, Q50, Q100 and Q500 were provided by MaineDOT.

The USGS report stated earlier, reported flows for various years and determined peak flows.

The report also stated that flows released from Sebago Lake were insignificant. Which suggests that the small to intermediate tributaries significantly contributed to the peak flow. The calculated and reported values are shown.

Table 3. Recurrence intervals, exceedance probabilities, and peak flows for the Presumpscot River at the S.D. Warren Bridge in Westbrook, Maine (site 10) [cubic feet per second abbreviated as ft^3/s]

Recurrence interval	Exceedance probability	Peak flow (ft^3/s)
2 years	0.500	5,310
5 years	0.200	7,830
10 years	0.100	9,890
25 years	0.040	13,000
50 years	0.020	15,700
100 years	0.010	18,900
200 years[a]	0.005	22,400
500 years[a]	0.002	28,000

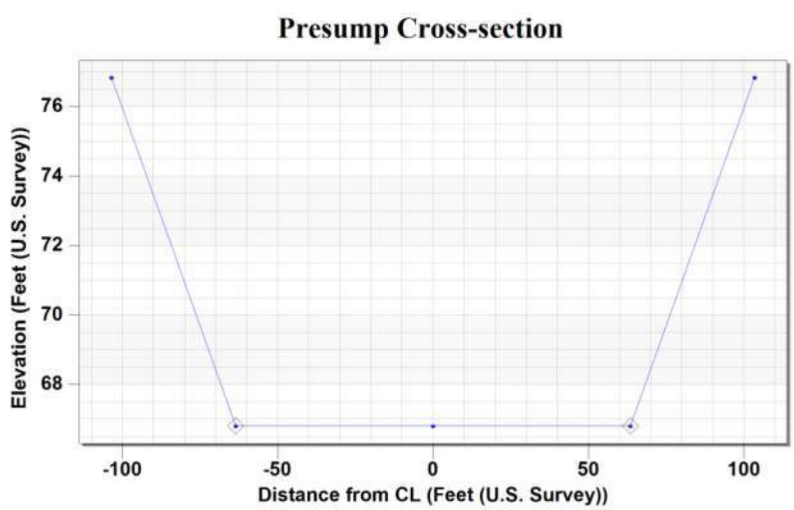
[a] Recurrence intervals greater than 100 years have a large uncertainty associated with them.

Table 5. Significant historical peak flows on the Presumpscot River in Westbrook, Maine

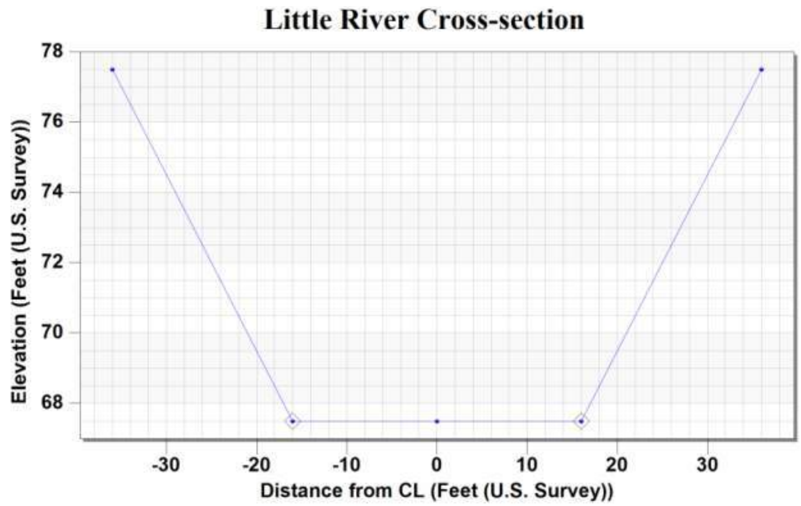
Date	Flow (cubic feet per second)
April 15, 1895	13,000[a]
March 1, 1896	13,800[a]
February 13, 1900	11,300[a]
March 2, 1900	9,720[a]
May 17, 1916	12,400[a]
June 17, 1917	9,710[a]
March 12, 1936	11,200[a]
September 11, 1954	12,400[a]
March 14, 1977	11,200[b]
May 12, 1989	9,200[c]
August 20, 1991	13,900[c]
October 22, 1996	23,300[d]

General Model Development

1. **Surface Generation**
    a. Conventional survey was completed by MaineDOT for this project and incorporated as a DTM file. This file encompassed stream bathymetry but did not adequately encompass the floodplain.
        i. A LiDAR surface was used to supplement the on-the-ground survey to incorporate missing floodplain areas.
        ii. DEM/LiDAR panels were downloaded from the NOAA Data Access Viewer Website. https://coast.noaa.gov/dataviewer/#/LiDAR/search/
    b. The geometric data in SMS included:
        i. Survey data.
        ii. LiDAR data.
    c. SMS's feature stamping tool was used to generate additional bathymetry.
        i. Presumpscot River
            - Slope was determined using information regarding the Mallison Falls Dam is found on the Low Impact Hydropower's institute website and existing topography/bathymetry at the Saccarappa Dam provided by Sappi (Dam Owner)
            - Tailwater elevation 71-ft (NGVD29) taken from Low Impact Hydropower Institute.
            - Invert elevation near Saccarappa Dam is approximately 66 ft +/- (NGVD29, Topo attached)
            - Slope: (71ft-66ft)/10464.48ft = 0.00478 ft/ft, note for this study, these elevations were used to define the Presumpscot River
            - The following cross-section was assumed



        ii. Little River
            - Upstream of the crossing, a slope of 0.005 ft/ft was assumed
            - Downstream of the crossing, the river bed was assumed to be relatively flat.
            - The following cross-section was assumed



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2. Mesh Development

a. The model domain was manually defined for both the existing and proposed conditions.

b. Triangular elements typically were used to model floodplain and tributaries.

c. Quadrilateral mesh elements typically were used to develop the mesh within the river.

3. Calibration Process

a. Since field measurements were not obtained, USGS Water-Resources Investigation Report 97-4189, titled "Flood of October 1996 in Southern Maine" was utilized for calibration purposes.
   i. This report documented peak water surface elevations at various site on the Presumpscot and Little River. These water surface elevations were reported in NGVD29.
   ii. The "Online Vertical Datum Transformation" website was used to determine the conversion from NGVD29 to NAVD88.
      - https://www.ngs.noaa.gov/cgi-bin/VERTCON/vert_con.prl
      - The difference in elevation from NGVD29 to NAVD88 at the Saccarappa Dam is -0.692ft.
      - The table below shows peak water surface elevations from the 1996 flood that were used to calibrate the model

Location	WSE (NGVD)	WSE (NAVD88)
Upstream of Saccarappa Dam	76.8 ft	76.1 ft
7 Rousseau Road, Windham	80.9 ft	80.21 ft
Upstream of Route 237 Bridge	88.6 ft	87.91 ft
Downstream of Route 237 Bridge	87.3 ft	86.61 ft
Mallison St. Bridge, Gorham (75.5 ft Downstream)	95.5 ft	94.81 ft

b. <u>Peak Flow during 1996</u>
   i. Based on the report, the peak flow just downstream of the Saccarappa Dam was approximately **23,300 cfs**.
   ii. The upstream boundary conditions had a constant discharge and subcritical regime. Therefore, an Inlet-Q type condition was used.

c. <u>Upstream Boundary Condition – Little River</u>
   i. Based on the report, there was approximately 17-inches of rain that fell during a 36-hour period (10/20-10/22 of 1996).
   ii. Based on streamstats, the upper limit confidence interval for the 500-year return is approximately **9,340 cfs**. Based on a drainage area of 49 sq. miles, this seems reasonable for a rainfall of 15-inches during a 48-hour event.

d. <u>Additional Internal Boundary Conditions</u>
   i. Internal Sink Boundary Conditions were used to add flow to the model domain downstream of the Little River Confluence at the Mosher Brook and Inkhorn Brook confluences assuming a 500-year return per streamstats.
      - Mosher Brook – **336 cfs**
      - Inkhorn Brook – **752 cfs**

e. <u>Upstream Boundary Condition – Presumpscot River</u>

i. Information regarding the Mallison Falls Dam is found on the Low Impact Hydropower's institute website and has a crest length of 280 ft +/-. Pertinent information regarding this dam is attached. The 1996 report stated a headwater elevation of approximately 95.5 ft (NGVD29). The Mallison Falls Dam has a crest elevation of approximately 90.5 ft (NGVD29). The following weir equation was used to determine the flow over dam during 1996 assuming an additional 0.39 ft (5fps) of velocity head.
- $Q = 3.2LH^{3/2}$
- $Q = 3.2 * 280 * 5.39^{3/2} = 11{,}212 \text{ cfs}$

ii. Just downstream of the Mallison Falls Dam, the Colley Wright Brook is found and assuming a 500-year return, a flow **1,250 cfs** was calculated using streamstats.

iii. The Mallison Falls Dam has a maximum turbine capacity of **390 cfs**.

iv. A total flow of 12,852 cfs was calculated for the Upstream Boundary Condition Flow but a flow of <u>**12,872 cfs**</u> was used for mass conservation.

f. <u>Downstream Boundary Condition</u>

i. Based on the 1996 Report, approximately 1500-ft upstream of the dam, the peak water surface elevation of 76.1 ft was obtained and was used to define the tailwater elevation of the Presumpscot River.

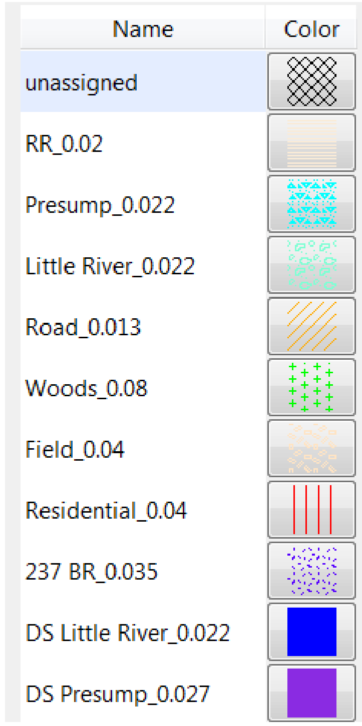
ii. Subcritical flow is anticipated for the majority of the events. Therefore, the downstream boundary condition is an Exit-H type and the adjusted Water Surface Elevations are entered for different storm events.

g. <u>Low Chord Elevation</u>

i. The superstructure is sloped but for this project the lowest value was used to define a constant low chord at this site. A low chord elevation of 84.11 ft, NAVD88 (84.78 ft, NGVD29)

4. Material Properties and Roughness Values

a. Initial values that produced good results.



Name	Color
unassigned	
RR_0.02	
Presump_0.022	
Little River_0.022	
Road_0.013	
Woods_0.08	
Field_0.04	
Residential_0.04	
237 BR_0.035	
DS Little River_0.022	
DS Presump_0.027	

5. Calibration Sensitivity

a. The purpose of this sensitivity analysis was to show how the river system reacts to changes in flows and n values.

Model Name	Description
Cal 1	Produce results within 0.15ft using boundary conditions and material properties stated above. This mesh for this model was used for this sensitivity analysis.
Cal 2	• Upstream BC for Presumpscot inlet was increased by 2,000 cfs. • Upstream BC for Little River was decreased by 2,000 cfs. • Presump n value increased to 0.025 • DS_Presump n value increased to 0.029
Cal 3	• Removed Internal Sink Boundary Conditions • Upstream BC for Presumpscot inlet was increased to 17,960 cfs

	• Upstream BC for Little River was decreased to 5,340 cfs • n values remained the same as Cal 2
Cal 4	• Removed Internal Sink Boundary Conditions • Upstream BC for Presumpscot inlet was increased to 18,790 cfs • Upstream BC for Little River was decreased to 4,340 cfs • n values that defined Presumpscot and Little River were increased to 0.03

Calibration Results

Cal -1			
Location	**Observed Value**	**Computed Value**	**Difference**
DS 237	86.61	86.642	0.032
US 237	87.91	87.844	-0.066
7 Rousseau Rd	80.21	80.074	-0.136

Cal -2			
Location	**Observed Value**	**Computed Value**	**Difference**
DS 237	86.61	86.709	0.099
US 237	87.91	87.444	-0.466
7 Rousseau Rd	80.21	80.075	-0.135

Cal -3			
Location	**Observed Value**	**Computed Value**	**Difference**
DS 237	86.61	86.859	0.249
US 237	87.91	87.243	-0.667
7 Rousseau Rd	80.21	79.907	-0.303

Cal -4			
Location	**Observed Value**	**Computed Value**	**Difference**
DS 237	86.61	87.479	0.869
US 237	87.91	87.758	-0.152
7 Rousseau Rd	80.21	80.183	-0.027

6. **Rating Curve Development**
    a. The purpose of developing the rating curve for the Saccarappa Dam is to determine the water surface elevation upstream of the dam for various flow. The water surface elevation is used as the downstream boundary condition for the hydraulic models; this assumes that the water surface elevation is approximately the same 1500' upstream of the dam.
    b. The Engineering Manager, Barry Stemm, for the Saccarappa Dam provided the existing site plan for the dam. He also provided a top of spillway elevation of 70 ft +/- 0.2 ft (NGVD29).
    c. The historic data provided in Table 7 of the USGS Water-Resources Investigation Report 97-4189 was used to determine the weir coefficient for the dam.
    d. Assume dam efficiency is maintained below 12,400 cfs and decreases linearly above 12,400 cfs.
    e. The rating curve is then developed using the broad crested weir equation. The rating curve is used to determine the water surface elevation of the desired flow event for the downstream boundary condition.

7. **Check Existing Calibrated Model**
    a. The USGS Scientific Investigations Report 2009-5102, titled "Flood of April 2007 in Southern Maine" was utilized to check the calibrated model.
        i. This report documented peak water surface elevations at various site on the Presumpscot and Little River. These water surface elevations were reported in NAVD88.
        ii. The table below shows peak water surface elevations from the 2007 flood that were used to calibrate the model

Location	WSE (NAVD88)
Upstream of Route 237 Bridge	81.80 ft
Downstream of Route 237 Bridge	81.69 ft

    b. <u>Peak Flow during 2007</u>
        i. A stream gage (01064118) is located downstream of the Saccarappa Dam. Data from the stream gage and National Weather Service information were used to develop a stage discharge graph.
        ii. The stage discharge graph was used to determine that the flow near the Saccarappa Dam during the 2007 flood was about **11,825 cfs**.
    c. <u>Upstream Boundary Condition – Little River</u>
        i. Two models were run to test the sensitivity of the water surface elevation by varying the flow in the Little River.
        ii. The first model used a flow based on streamstats upper limit confidence interval for the 2-year return event; this is approximately **2,040 cfs**.

iii. The second model used a flow based on streamstats upper limit confidence interval for the 25-year return event; this is approximately **4,950 cfs**.

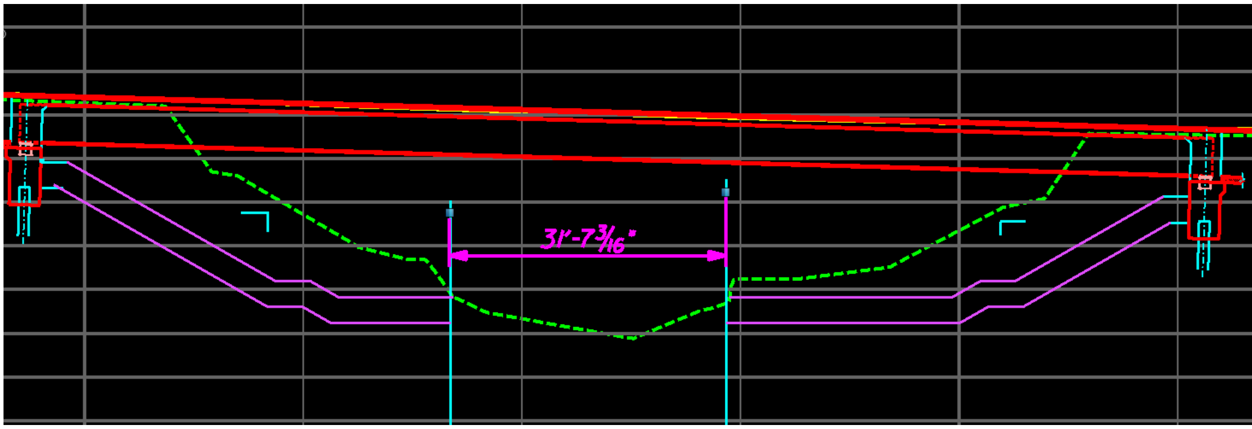
d. Additional Internal Boundary Conditions
   i. The Internal Sink Boundary Conditions were used to add flow to the model domain downstream of the Little River Confluence at the Mosher Brook and Inkhorn Brook confluences assuming a 2-year return per streamstats.
      - Mosher Brook – **68 cfs**
      - Inkhorn Brook – **153 cfs**

e. Upstream Boundary Condition – Presumpscot River
   i. The upstream boundary condition for the Presumpscot River was determined using mass conservation.
   ii. For the first model, the constant inflow had to be **9,564 cfs**.
   iii. For the second model, the constant inflow had to be **6,654 cfs**.

f. Downstream Boundary Condition
   i. Based on the 11,825 cfs flow and using the Saccarappa Dam rating curve, the water surface elevation was determined to be 72.99 ft.
   ii. Subcritical flow is anticipated for this event events. Therefore, the downstream boundary condition is an Exit-H type.

8. Proposed Conditions

a. The following section was integrated into the existing surface. Elevations and Distances were measured in CAD.



9. Q50 and Q100 Proposed Analysis without dam

a. The purpose of this analysis was to show or not show if there was an effect on the proposed condition water surface elevations if the dam was removed.

b. Normal Slope of 0.00478, Composite n of 0.045
   i. DS BC Q50 WSEL 65.6 ft
   ii. DS BC Q100 WSEL 66.61 ft

10. Scour Analysis

a. Refer to MaineDOT BDG 2.3.11.1

b. Analyze the Little River without the influence of the Saccarappa Dam
   i. Normal slope of 0.00478, Composite n of 0.035

c. Neglect Mosher Brook and Inkhorn Brook for scour analysis
d. Largest Scour Potential will occur when there is minimal tailwater produced from Presumpscot
    i. Note: Based on the FIS and USGS April 2007 report, Presumpscot Dam is at peak flow and Little River is a low flow.
e. 2-year scour analysis
    i. Assume @ Dam – 3,040 cfs
    ii. Little River – 2,040 cfs
    iii. US Presumpscot – 1,000 cfs
    iv. DS WSEL - 58.86 ft
f. 5-year scour analysis
    i. 2-year @ dam – 5,310 cfs
    ii. Little River – 3,070 cfs
    iii. US Presumpscot – 2,240 cfs
    iv. DS WSEL - 60.19 ft
g. 10-year scour analysis
    i. 2-year @ Dam – 5,310 cfs
    ii. Little River – 3,860 cfs
    iii. US Presumpscot – 1,450 cfs
    iv. DS WSEL - 60.19 ft
h. 25-year Scour analysis
    i. 5-year @ Dam – 7,830 cfs
    ii. Little River – 4,950 cfs
    iii. US Presumpscot – 2,880 cfs
    iv. DS WSEL - 61.39 ft
i. 50-year Scour Analysis
    i. 10-year @ Dam – 9,890 cfs
    ii. Little River – 5,840 cfs
    iii. US Presumpscot – 4,050 cfs
    iv. DS WSEL - 62.25 ft
j. 100-year Scour Analysis
    i. 10 year @ Dam – 9,890 cfs
    ii. Little River – 6,820 cfs
    iii. US Presumpscot – 3,070 cfs
    iv. DS WSEL - 62.25 ft
k. 200-year Scour Analysis
    i. 25-year @ Dam – 13,000 cfs
    ii. Little River – 8,015 cfs
    iii. US Presumpscot – 4,985 cfs
    iv. DS WSEL - 63.39 ft
l. 500-year Scour Analysis
    i. 25-year @ Dam 13,000 cfs
    ii. Little River – 9,410 cfs
    iii. US Presumpscot – 3,590 cfs
    iv. DS WSEL - 63.39 ft

Orthometric Height Conversion

Orthometric height conversion is performed by calculating the datum shift based from modeled values.
The resulting datum shift is displayed.
The converted orthometric height is displayed only if the height to be converted from was not left blank.
******* See input format details below *******

Latitude and Longitude within the Contiguous United States are REQUIRED:

```
Positions may be entered in any of the following three formats:
  1. degrees, minutes and decimal seconds (including leading zeros)
     Lon: (XXX XX XX.XXX)           Lat: (XX XX XX.XXX)
     Lon: 098 33 23.232  good       Lat: 45 33 23.232  good
     Lon:  98 33 23.232  bad        Lat:  5 33 23.232  bad
     Lon: 098 03 23.342  good       Lat: 45 03 03.232  good
     Lon: 098  3 23.342  bad        Lat: 45  3  3.232  bad
  2. degrees and decimal minutes (including leading zeros)
     Lon: (XXX XX.XXX)              Lat: (XX XX.XXX)
     Lon: 098 23.232  good          Lat: 45 33.232  good
     Lon:  98 23.232  bad           Lat:  5 23.232  bad
     Lon: 098 03.342  good          Lat: 45 03.232  good
     Lon: 098  3.342  bad           Lat: 45  3.232  bad
  3. decimal degrees (including leading zeros)
     Lon: (XXX.XXX)                 Lat: (XX.XXX)
     Lon: 098.232  good             Lat: 45.232  good
     Lon:  98.232  bad              Lat:  5.232  bad

Note: There MUST be one or more blanks between entry fields
      Decimals can be keyed commensurate with the field's precision, but are not required.
```

Orthometric Height to be converted FROM is OPTIONAL:

```
Height may be entered in either meters or U.S. survey feet:
  1. meters:  xxxx.xxx
  2. feet  :  xxxx.xx  FT  ( MUST include FT or ft for feet !)
```

ENTER North Latitude :........ 43.679

ENTER West Longitude :....... 070.369

ENTER Orthometric Height : 70.0 FT **-- Entry is Optional; Default units (meters) --**

SELECT Vertical Datum :... (•) **NGVD 29** ( ) **NAVD 88 -- of the entered height --**

If the orthometric height is unknown DO NOT ENTER ZERO; leave the entry field BLANK !

Submit Clear

It is possible that the datum shift computation may not succeed.
There could be many reasons for this, including position and height entry errors. In such a case, an error code is returned.
Questions concerning the datum shift computation process may be mailed to NGS

Questions concerning the VERTCON process may be mailed to [NGS]

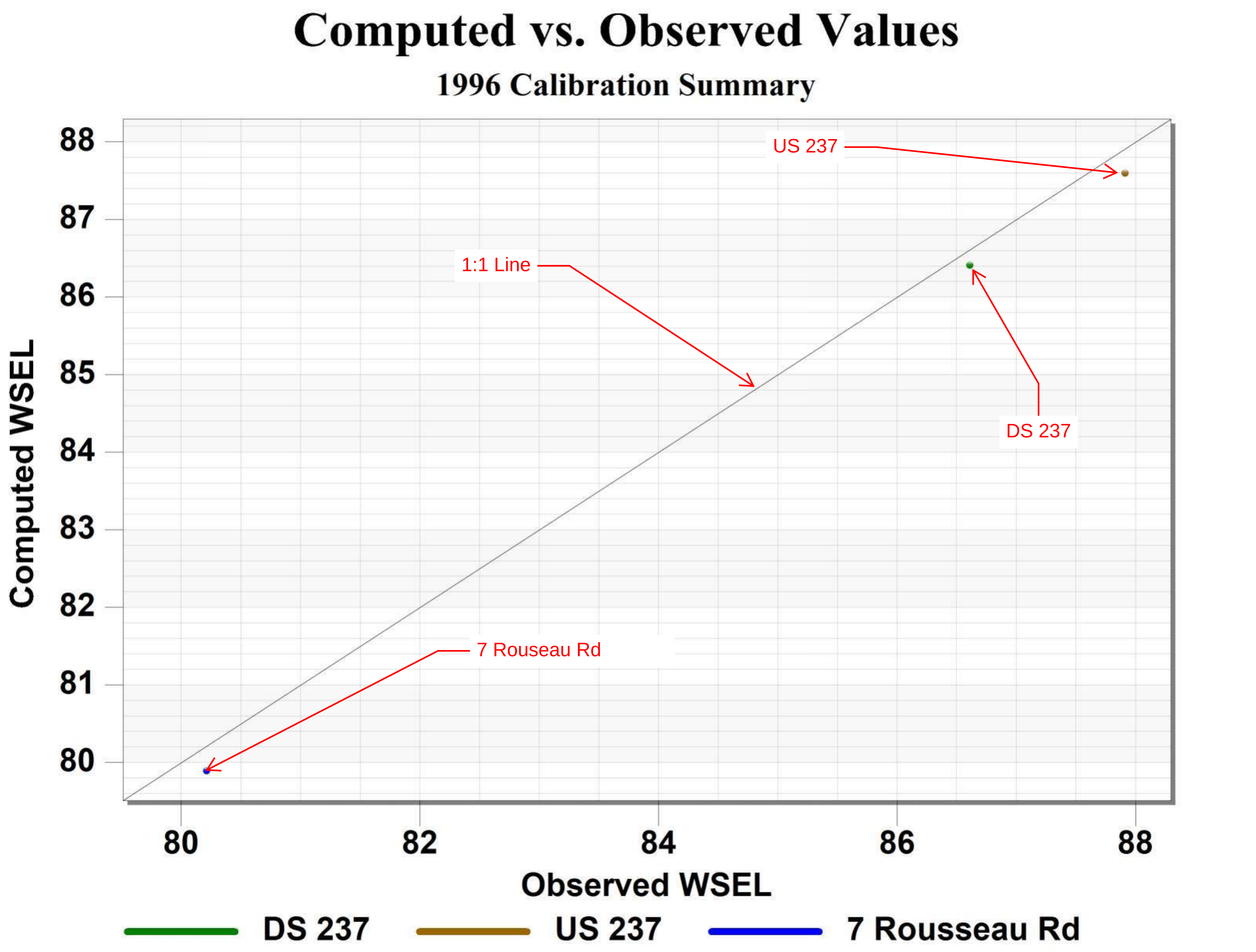
Latitude: 43.679

Longitude: 070.369

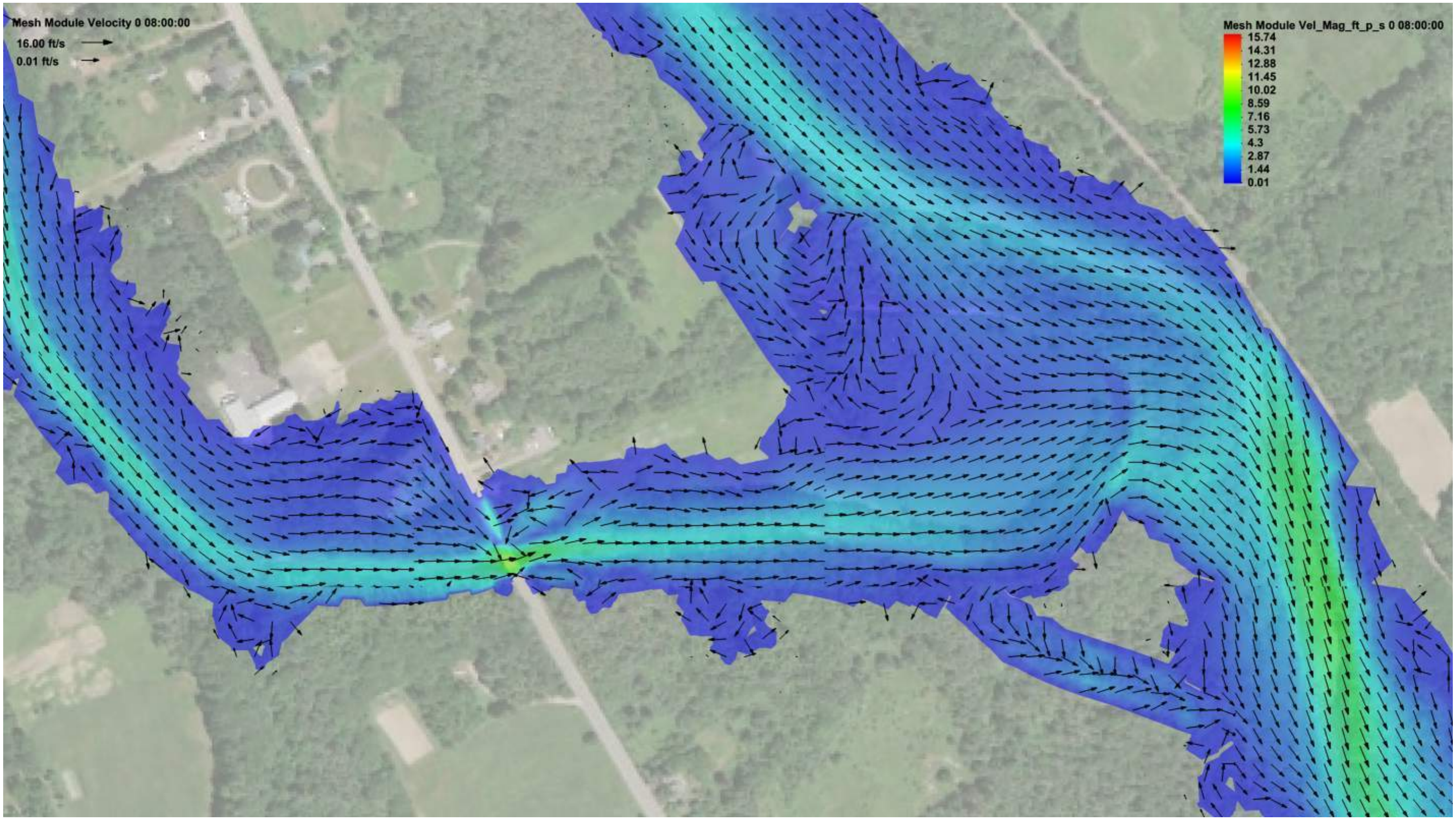
NGVD 29 height: 70.0 FT

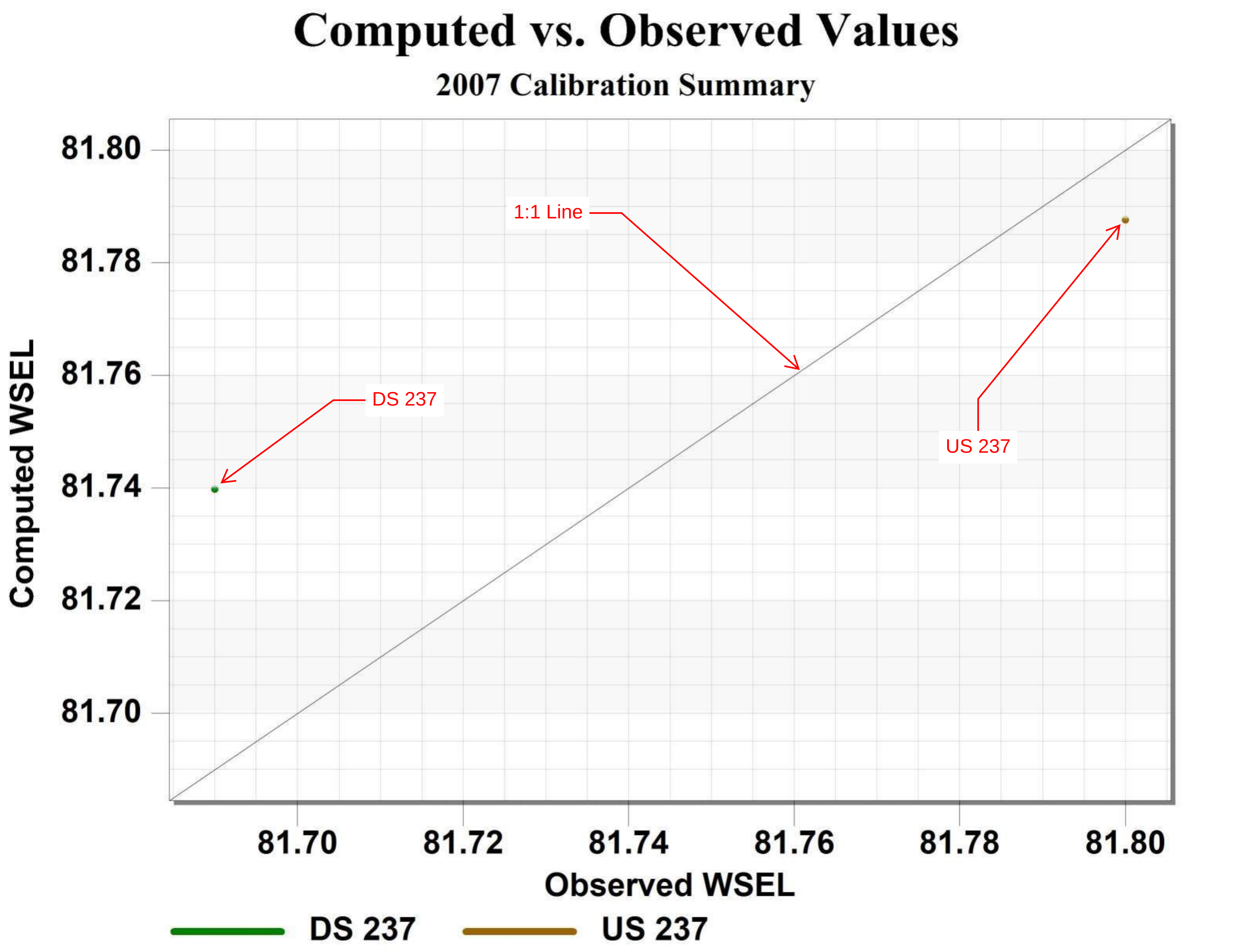
Datum shift(NAVD 88 minus NGVD 29): -0.692 feet

Converted to NAVD 88 height: 69.308 feet


Computed vs. Observed Values
1996 Calibration Summary
Computed WSEL
Observed WSEL
80
81
82
83
84
85
86
87
88
80
82
84
86
88
US 237
1:1 Line
DS 237
7 Rousseau Rd
DS 237
US 237
7 Rousseau Rd

Existing Conditions: 1996 Calibration Velocity Profile

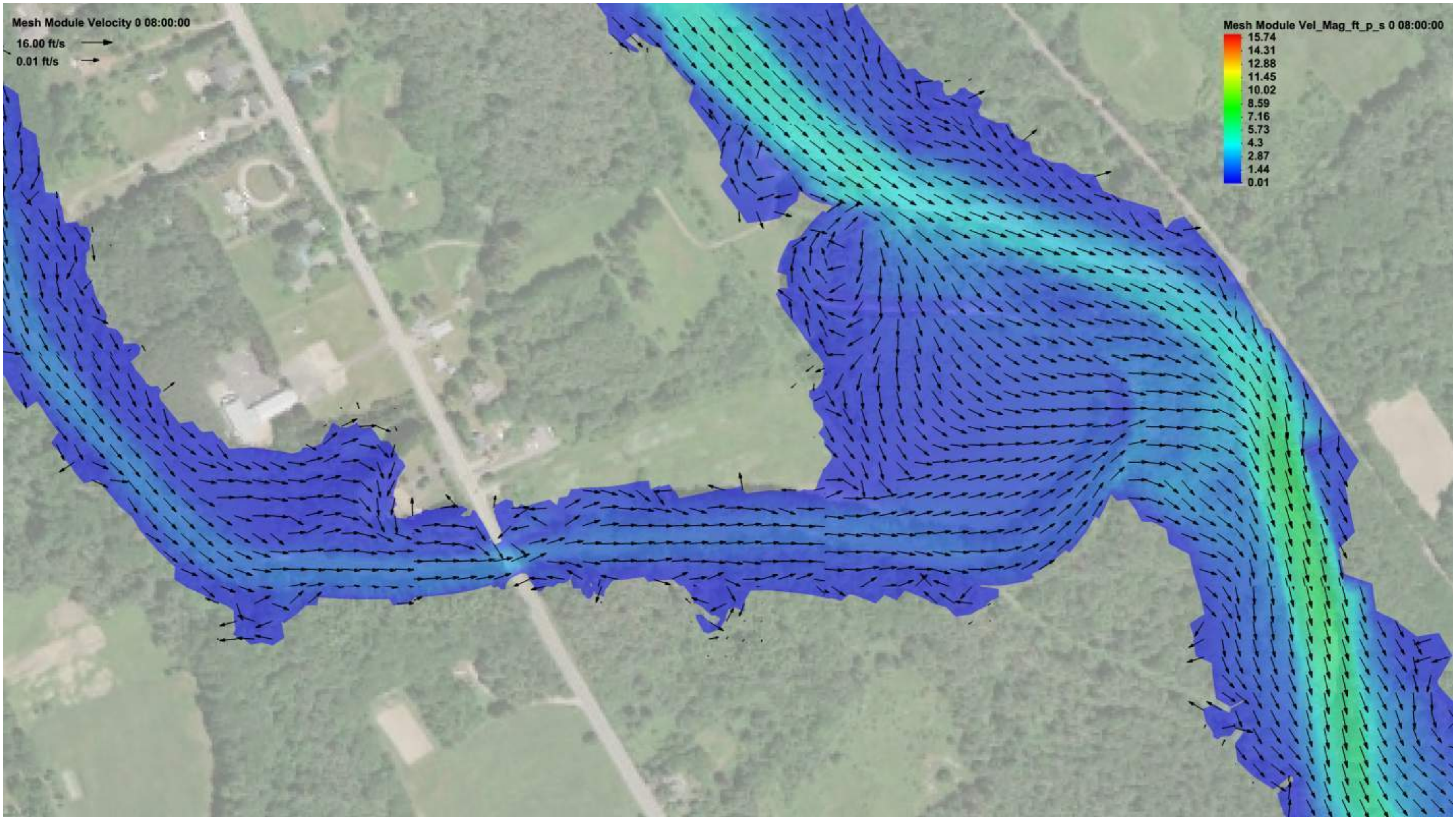




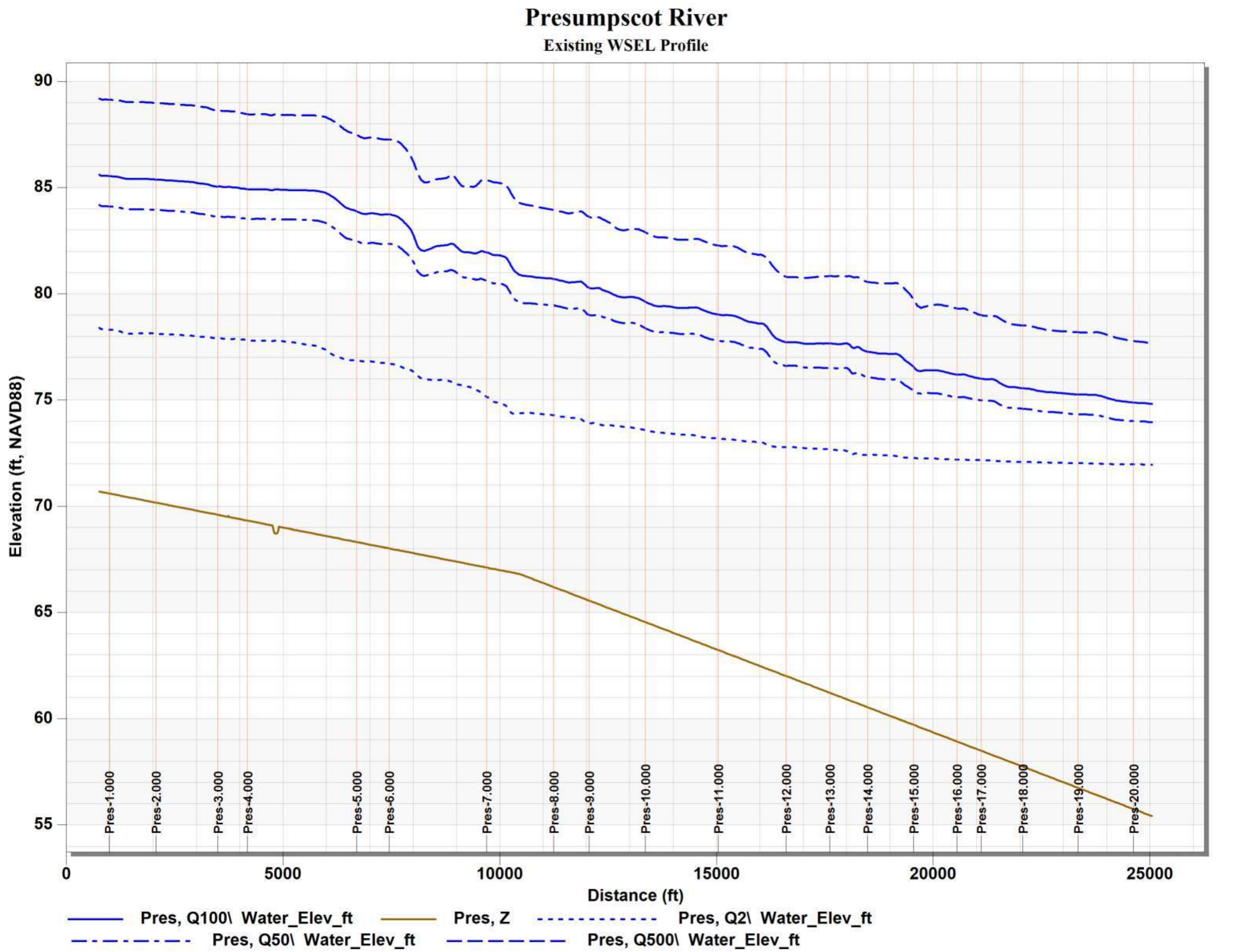
Computed vs. Observed Values
2007 Calibration Summary
Computed WSEL
81.80
81.78
81.76
81.74
81.72
81.70
1:1 Line
DS 237
US 237
81.70
81.72
81.74
81.76
81.78
81.80
Observed WSEL
DS 237
US 237

Hoyle, Tanner Project No. 923403.08
Little River Bridge, Gorham, ME
Route 237 over the Little River
MaineDOT WIN 023909.00

Existing Conditions: 2007 Calibration Velocity Profile



Presumpscot River 1D Cross-Sections



Presumpscot River
Existing WSEL Profile
Elevation (ft, NAVD88)
Distance (ft)
90
85
80
75
70
65
60
55
0
5000
10000
15000
20000
25000
Pres-1.000
Pres-2.000
Pres-3.000
Pres-4.000
Pres-5.000
Pres-6.000
Pres-7.000
Pres-8.000
Pres-9.000
Pres-10.000
Pres-11.000
Pres-12.000
Pres-13.000
Pres-14.000
Pres-15.000
Pres-16.000
Pres-17.000
Pres-18.000
Pres-19.000
Pres-20.000
Pres, Q100\ Water_Elev_ft
Pres, Z
Pres, Q2\ Water_Elev_ft
Pres, Q50\ Water_Elev_ft
Pres, Q500\ Water_Elev_ft

Existing: 1996 Cal\Calibration

		Z	*Froude*	*Water_Elev_ft*	*Vel_Mag_ft_p_s*	
Reach	Station	Min	Ave	Ave	Max	Ave
Presump	1	70.61	0.06	87.36	3.29	1.16
Presump	2	70.17	0.06	87.06	3.64	1.1
Presump	3	69.6	0.08	86.79	3.98	1.36
Presump	4	69.32	0.05	86.68	4.15	0.93
Presump	5	68.32	0.08	86.39	7.25	1.23
Presump	6	68.02	0.06	85.66	6.04	1.15
Presump	7	67.12	0.12	83.05	6.65	1.55
Presump	8	66.2	0.08	82.48	6.89	1.5
Presump	9	65.55	0.18	82.15	6.8	2.63
Presump	10	64.55	0.14	81.19	7.22	2.39
Presump	11	63.24	0.12	80.61	6.2	2.13
Presump	12	62.01	0.14	79.26	7.75	2.33
Presump	13	61.23	0.13	79.14	5.7	1.81
Presump	14	60.54	0.13	78.78	5.29	1.72
Presump	15	59.72	0.24	78.16	8.11	3.49
Presump	16	58.93	0.11	77.82	6.76	1.73
Presump	17	58.5	0.08	77.49	6.61	1.18
Presump	18	57.75	0.06	77.01	6.49	1
Presump	19	56.75	0.07	76.56	5.48	1.01
Presump	20	55.75	0.08	76.27	5.86	1.27

Existing: 2007 Cal\Calibration

		Z	*Froude*	*Water_Elev_ft*	*Vel_Mag_ft_p_s*	
Reach	Station	Min	Ave	Ave	Max	Ave
Presump	1	70.61	0.08	82.80	3.68	1.22
Presump	2	70.17	0.07	82.48	4.29	1.13
Presump	3	69.6	0.09	81.97	5.04	1.36
Presump	4	69.32	0.06	81.90	4.81	1.03
Presump	5	68.32	0.05	80.76	6.63	0.76
Presump	6	68.02	0.06	80.56	5.42	0.91
Presump	7	67.12	0.11	78.55	6.52	1.19
Presump	8	66.2	0.09	77.86	5.63	1.20
Presump	9	65.55	0.14	77.51	5.83	1.79
Presump	10	64.55	0.13	76.74	6.22	1.68
Presump	11	63.24	0.10	76.20	5.34	1.46
Presump	12	62.01	0.10	75.20	6.10	1.53
Presump	13	61.23	0.08	74.97	4.80	1.12
Presump	14	60.54	0.07	74.63	5.03	0.98
Presump	15	59.72	0.16	74.15	5.86	2.03
Presump	16	58.93	0.06	73.81	4.96	0.88
Presump	17	58.5	0.05	74.06	4.51	0.68
Presump	18	57.75	0.04	73.57	4.62	0.51
Presump	19	56.75	0.04	73.23	4.00	0.47
Presump	20	55.75	0.04	73.10	3.97	0.64

Existing: Q2\Dam In Place

		Z	*Froude*	*Water_Elev_ft*	*Vel_Mag_ft_p_s*	
Reach	Station	Min	Ave	Ave	Ave	Max
Presump	1	70.61	0.05	78.57	0.59	2.15
Presump	2	70.17	0.05	78.11	0.60	2.54
Presump	3	69.6	0.07	77.95	0.75	2.79
Presump	4	69.32	0.05	77.81	0.53	2.87
Presump	5	68.32	0.04	76.85	0.50	5.03
Presump	6	68.02	0.06	76.60	0.69	4.37
Presump	7	67.12	0.06	75.26	0.55	6.09
Presump	8	66.2	0.07	74.30	0.79	4.13
Presump	9	65.55	0.09	74.00	1.02	4.46
Presump	10	64.55	0.07	73.58	0.90	4.06
Presump	11	63.24	0.06	73.31	0.72	3.44
Presump	12	62.01	0.06	72.81	0.78	3.41
Presump	13	61.23	0.03	72.64	0.46	2.95
Presump	14	60.54	0.03	72.53	0.38	3.12
Presump	15	59.72	0.05	72.33	0.83	3.27
Presump	16	58.93	0.03	72.35	0.44	2.57
Presump	17	58.5	0.02	72.38	0.34	2.35
Presump	18	57.75	0.02	72.08	0.25	2.33
Presump	19	56.75	0.02	72.11	0.22	2.04
Presump	20	55.75	0.02	71.98	0.30	1.94

Existing: Q50\Dam In Place

		Z	*Froude*	*Water_Elev_ft*	*Vel_Mag_ft_p_s*	
Reach	Station	Min	Ave	Ave	Max	Ave
Presump	1	70.61	0.07	84.14	3.05	1.05
Presump	2	70.17	0.05	83.98	3.49	0.97
Presump	3	69.6	0.07	83.66	3.98	1.16
Presump	4	69.32	0.05	83.56	3.91	0.82
Presump	5	68.32	0.06	82.49	6.96	0.80
Presump	6	68.02	0.06	82.24	5.72	1.00
Presump	7	67.12	0.11	80.21	6.43	1.24
Presump	8	66.2	0.09	79.90	6.06	1.37
Presump	9	65.55	0.15	78.95	6.17	1.99
Presump	10	64.55	0.14	78.31	6.63	1.95
Presump	11	63.24	0.11	77.85	5.69	1.72
Presump	12	62.01	0.11	76.63	6.84	1.76
Presump	13	61.23	0.09	76.45	5.22	1.31
Presump	14	60.54	0.10	76.06	5.21	1.26
Presump	15	59.72	0.18	75.47	6.79	2.46
Presump	16	58.93	0.08	75.08	5.77	1.26
Presump	17	58.5	0.06	75.24	5.41	0.87
Presump	18	57.75	0.05	74.65	5.48	0.72
Presump	19	56.75	0.06	74.30	4.72	0.71
Presump	20	55.75	0.06	74.17	4.93	0.85

Existing: Q100\Dam In Place

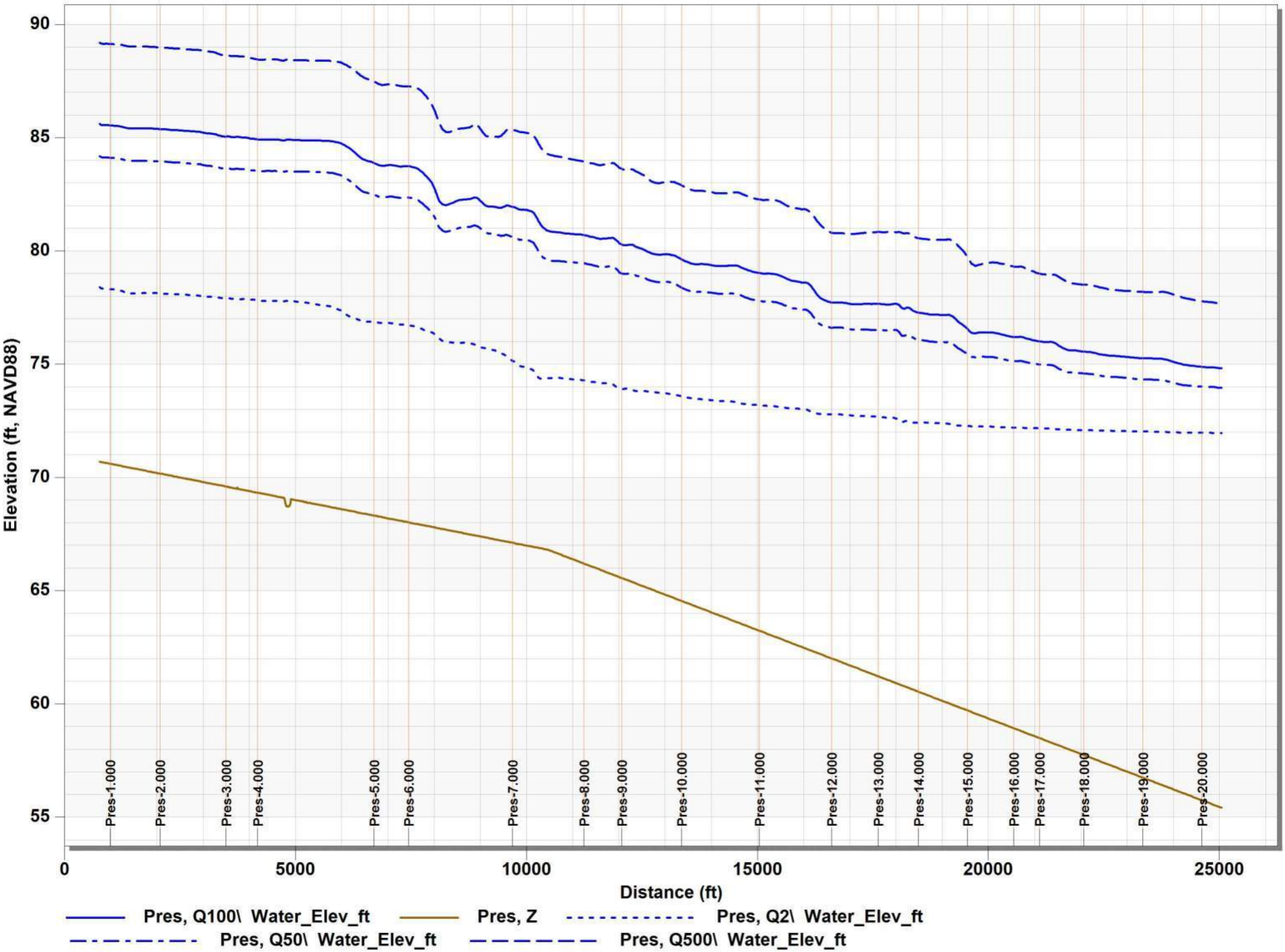
Reach	Station	*Z* Min	*Froude* Ave	*Water_Elev_ft* Ave	*Vel_Mag_ft_p_s* Max	Ave
Presump	1	70.61	0.07	85.57	3.30	1.16
Presump	2	70.17	0.06	85.40	3.71	1.06
Presump	3	69.6	0.09	85.09	4.18	1.32
Presump	4	69.32	0.05	84.94	4.22	0.91
Presump	5	68.32	0.07	84.87	7.14	1.03
Presump	6	68.02	0.06	83.96	5.90	1.07
Presump	7	67.12	0.10	81.44	6.51	1.39
Presump	8	66.2	0.09	81.04	6.41	1.51
Presump	9	65.55	0.20	80.22	6.43	2.32
Presump	10	64.55	0.14	79.56	6.89	2.18
Presump	11	63.24	0.12	79.07	5.97	1.92
Presump	12	62.01	0.13	77.73	7.32	2.07
Presump	13	61.23	0.12	77.69	5.50	1.58
Presump	14	60.54	0.12	77.23	5.29	1.52
Presump	15	59.72	0.18	76.72	7.48	2.75
Presump	16	58.93	0.09	76.13	6.28	1.46
Presump	17	58.5	0.07	76.11	6.01	1.02
Presump	18	57.75	0.06	75.58	6.01	0.84
Presump	19	56.75	0.07	75.22	5.13	0.86
Presump	20	55.75	0.07	75.05	5.46	1.02

Existing: Q500\Dam In Place

Reach	Station	*Z* Min	*Froude* Ave	*Water_Elev_ft* Ave	*Vel_Mag_ft_p_s* Ave	Max
Presump	1	70.61	0.07	89.19	1.42	3.90
Presump	2	70.17	0.06	88.97	1.35	4.18
Presump	3	69.6	0.10	88.96	1.61	4.53
Presump	4	69.32	0.06	88.71	1.14	4.81
Presump	5	68.32	0.11	88.10	1.43	7.22
Presump	6	68.02	0.06	87.30	1.25	6.07
Presump	7	67.12	0.12	84.87	1.76	6.78
Presump	8	66.2	0.09	84.12	1.72	7.27
Presump	9	65.55	0.17	83.72	2.83	7.25
Presump	10	64.55	0.15	83.09	2.60	7.51
Presump	11	63.24	0.12	82.39	2.28	6.39
Presump	12	62.01	0.15	80.80	2.73	8.08
Presump	13	61.23	0.13	80.67	1.97	5.84
Presump	14	60.54	0.13	80.47	1.95	5.17
Presump	15	59.72	0.24	79.77	3.88	8.62
Presump	16	58.93	0.11	79.36	1.86	7.03
Presump	17	58.5	0.09	79.00	1.36	6.90
Presump	18	57.75	0.07	78.53	1.12	6.74
Presump	19	56.75	0.07	78.13	1.13	5.61
Presump	20	55.75	0.09	77.80	1.48	5.94

Presumpscot River

Proposed WSEL Profile



Existing: Q2\Dam In Place

		Z	*Froude*	*Water_Elev_ft*	*Vel_Mag_ft_p_s*	
Reach	Station	Min	Ave	Ave	Ave	Max
Presump	1	70.61	0.05	78.39	0.59	2.18
Presump	2	70.17	0.05	78.11	0.60	2.54
Presump	3	69.6	0.07	77.95	0.70	2.80
Presump	4	69.32	0.05	77.81	0.53	2.87
Presump	5	68.32	0.04	76.94	0.50	5.03
Presump	6	68.02	0.06	76.60	0.69	4.37
Presump	7	67.12	0.06	75.26	0.55	6.09
Presump	8	66.2	0.07	74.30	0.79	4.13
Presump	9	65.55	0.09	74.00	1.02	4.46
Presump	10	64.55	0.07	73.58	0.90	4.06
Presump	11	63.24	0.06	73.31	0.72	3.44
Presump	12	62.01	0.06	72.81	0.78	3.41
Presump	13	61.23	0.03	72.68	0.46	2.95
Presump	14	60.54	0.03	72.61	0.38	3.12
Presump	15	59.72	0.05	72.33	0.84	3.27
Presump	16	58.93	0.03	72.35	0.44	2.57
Presump	17	58.5	0.02	72.38	0.34	2.35
Presump	18	57.75	0.02	72.08	0.25	2.33
Presump	19	56.75	0.02	72.11	0.22	2.04
Presump	20	55.75	0.02	71.98	0.30	1.94

Existing: Q50\Dam In Place

		Z	*Froude*	*Water_Elev_ft*	*Vel_Mag_ft_p_s*	
Reach	Station	Min	Ave	Ave	Ave	Max
Presump	1	70.61	0.07	84.19	1.06	3.05
Presump	2	70.17	0.05	83.95	0.97	3.51
Presump	3	69.6	0.07	83.63	1.15	3.99
Presump	4	69.32	0.05	83.54	0.82	3.91
Presump	5	68.32	0.06	82.47	0.80	6.97
Presump	6	68.02	0.06	82.20	0.99	5.71
Presump	7	67.12	0.11	80.19	1.25	6.43
Presump	8	66.2	0.09	79.88	1.37	6.05
Presump	9	65.55	0.15	78.94	1.99	6.17
Presump	10	64.55	0.14	78.28	1.92	6.65
Presump	11	63.24	0.11	77.85	1.73	5.68
Presump	12	62.01	0.11	76.60	1.78	6.83
Presump	13	61.23	0.09	76.43	1.30	5.23
Presump	14	60.54	0.10	76.19	1.26	5.22
Presump	15	59.72	0.18	75.46	2.44	6.77
Presump	16	58.93	0.08	75.07	1.26	5.77
Presump	17	58.5	0.06	75.24	0.87	5.40
Presump	18	57.75	0.05	74.65	0.73	5.48
Presump	19	56.75	0.06	74.30	0.71	4.72
Presump	20	55.75	0.06	74.06	0.85	4.92

Existing: Q100\Dam In Place

		Z	*Froude*	*Water_Elev_ft*	*Vel_Mag_ft_p_s*	
Reach	Station	Min	Ave	Ave	Ave	Max
Presump	1	70.61	0.07	85.86	1.17	3.29
Presump	2	70.17	0.05	85.37	1.06	3.73
Presump	3	69.6	0.08	85.41	1.29	4.19
Presump	4	69.32	0.05	84.91	0.91	4.23
Presump	5	68.32	0.07	84.81	1.05	7.15
Presump	6	68.02	0.06	83.77	1.07	5.89
Presump	7	67.12	0.10	81.42	1.39	6.51
Presump	8	66.2	0.09	81.02	1.51	6.42
Presump	9	65.55	0.19	80.21	2.31	6.46
Presump	10	64.55	0.14	79.54	2.18	6.89
Presump	11	63.24	0.12	79.04	1.93	5.96
Presump	12	62.01	0.13	77.71	2.08	7.32
Presump	13	61.23	0.12	77.73	1.60	5.50
Presump	14	60.54	0.12	77.25	1.53	5.30
Presump	15	59.72	0.18	76.57	2.76	7.42
Presump	16	58.93	0.09	76.11	1.46	6.29
Presump	17	58.5	0.07	76.10	1.02	6.00
Presump	18	57.75	0.06	75.57	0.85	6.00
Presump	19	56.75	0.07	75.22	0.86	5.13
Presump	20	55.75	0.07	74.97	1.03	5.45

Existing: Q500\Dam In Place

		Z	*Froude*	*Water_Elev_ft*	*Vel_Mag_ft_p_s*	
Reach	Station	Min	Ave	Ave	Ave	Max
Presump	1	70.61	0.07	89.15	1.43	3.91
Presump	2	70.17	0.06	88.97	1.36	4.20
Presump	3	69.6	0.09	89.04	1.60	4.52
Presump	4	69.32	0.06	88.72	1.14	4.80
Presump	5	68.32	0.11	88.07	1.42	7.21
Presump	6	68.02	0.06	87.27	1.24	6.07
Presump	7	67.12	0.12	84.89	1.77	6.77
Presump	8	66.2	0.09	84.09	1.73	7.26
Presump	9	65.55	0.17	83.72	2.83	7.25
Presump	10	64.55	0.15	83.07	2.61	7.52
Presump	11	63.24	0.12	82.37	2.28	6.38
Presump	12	62.01	0.16	80.84	2.73	8.11
Presump	13	61.23	0.13	80.67	1.99	5.83
Presump	14	60.54	0.13	80.47	1.92	5.17
Presump	15	59.72	0.24	79.76	3.98	8.54
Presump	16	58.93	0.11	79.31	1.86	7.06
Presump	17	58.5	0.09	79.05	1.39	6.92
Presump	18	57.75	0.07	78.53	1.13	6.74
Presump	19	56.75	0.07	78.13	1.14	5.62
Presump	20	55.75	0.09	77.80	1.48	5.94

Existing: Q50\No Dam

		Z	*Froude*	*Water_Elev_ft*	*Vel_Mag_ft_p_s*	
Reach	Station	Min	Ave	Ave	Ave	Max
Presump	1	70.61	0.07	84.07	1.07	3.09
Presump	2	70.17	0.05	83.83	0.98	3.55
Presump	3	69.6	0.07	83.50	1.16	4.06
Presump	4	69.32	0.05	83.41	0.83	3.96
Presump	5	68.32	0.06	82.27	0.82	7.11
Presump	6	68.02	0.06	82.00	1.01	5.84
Presump	7	67.12	0.12	79.81	1.39	6.81
Presump	8	66.2	0.09	78.92	1.45	6.40
Presump	9	65.55	0.15	78.39	2.12	6.68
Presump	10	64.55	0.15	77.47	2.09	7.29
Presump	11	63.24	0.13	76.75	1.84	6.43
Presump	12	62.01	0.14	75.07	2.03	7.89
Presump	13	61.23	0.12	74.61	1.49	6.52
Presump	14	60.54	0.12	73.84	1.40	7.45
Presump	15	59.72	0.19	72.71	2.58	9.05
Presump	16	58.93	0.08	71.84	1.37	7.88
Presump	17	58.5	0.07	71.31	1.08	7.51
Presump	18	57.75	0.06	70.13	0.86	8.39
Presump	19	56.75	0.07	69.05	0.83	8.38
Presump	20	55.75	0.10	67.18	1.16	9.43

Existing: Q100\No Dam

		Z	*Froude*	*Water_Elev_ft*	*Vel_Mag_ft_p_s*	
Reach	Station	Min	Ave	Ave	Ave	Max
Presump	1	70.61	0.07	85.74	1.18	3.33
Presump	2	70.17	0.06	85.23	1.07	3.78
Presump	3	69.6	0.09	85.27	1.29	4.26
Presump	4	69.32	0.05	84.75	0.92	4.29
Presump	5	68.32	0.06	83.85	0.99	7.31
Presump	6	68.02	0.06	83.56	1.09	6.03
Presump	7	67.12	0.11	81.01	1.45	6.86
Presump	8	66.2	0.09	80.52	1.57	6.75
Presump	9	65.55	0.22	79.55	2.38	6.89
Presump	10	64.55	0.16	78.72	2.32	7.53
Presump	11	63.24	0.13	78.04	2.06	6.64
Presump	12	62.01	0.14	76.21	2.21	8.43
Presump	13	61.23	0.11	75.84	1.64	6.72
Presump	14	60.54	0.12	75.17	1.55	7.21
Presump	15	59.72	0.26	74.01	3.20	9.34
Presump	16	58.93	0.09	72.81	1.49	8.49
Presump	17	58.5	0.08	73.11	1.13	7.94
Presump	18	57.75	0.06	71.23	0.97	8.99
Presump	19	56.75	0.07	70.16	0.89	8.85
Presump	20	55.75	0.10	68.24	1.37	10.00

Existing: Q2\Scour

		Z	*Froude*	*Water_Elev_ft*	*Vel_Mag_ft_p_s*	
Reach	Station	Min	Ave	Ave	Ave	Max
Presump	1	70.61	0.04	76.25	0.30	1.18
Presump	2	70.17	0.03	76.03	0.30	1.30
Presump	3	69.6	0.04	76.19	0.32	1.39
Presump	4	69.32	0.02	75.96	0.20	1.43
Presump	5	68.32	0.04	75.10	0.36	4.54
Presump	6	68.02	0.06	74.54	0.52	4.26
Presump	7	67.12	0.04	73.08	0.43	5.55
Presump	8	66.2	0.06	71.48	0.70	4.34
Presump	9	65.55	0.11	71.20	1.17	5.20
Presump	10	64.55	0.09	70.19	0.89	4.65
Presump	11	63.24	0.07	68.90	0.71	4.17
Presump	12	62.01	0.08	67.55	0.92	4.43
Presump	13	61.23	0.05	66.88	0.53	4.26
Presump	14	60.54	0.05	66.24	0.50	4.38
Presump	15	59.72	0.10	65.01	1.19	4.71
Presump	16	58.93	0.07	64.30	0.62	4.35
Presump	17	58.5	0.04	63.88	0.45	4.44
Presump	18	57.75	0.04	63.15	0.38	4.51
Presump	19	56.75	0.04	62.11	0.37	4.69
Presump	20	55.75	0.06	60.61	0.55	5.06

Existing: Q5\Scour

		Z	*Froude*	*Water_Elev_ft*	*Vel_Mag_ft_p_s*	
Reach	Station	Min	Ave	Ave	Ave	Max
Presump	1	70.61	0.04	78.25	0.44	1.69
Presump	2	70.17	0.03	78.06	0.44	1.89
Presump	3	69.6	0.05	78.00	0.51	2.04
Presump	4	69.32	0.04	77.90	0.39	2.07
Presump	5	68.32	0.04	76.94	0.52	5.26
Presump	6	68.02	0.07	76.57	0.73	4.58
Presump	7	67.12	0.06	74.95	0.59	6.75
Presump	8	66.2	0.08	73.39	0.87	5.18
Presump	9	65.55	0.11	72.81	1.28	5.86
Presump	10	64.55	0.10	71.92	1.14	5.46
Presump	11	63.24	0.08	70.74	0.97	4.99
Presump	12	62.01	0.10	69.50	1.17	5.41
Presump	13	61.23	0.05	68.69	0.69	5.12
Presump	14	60.54	0.05	67.90	0.64	5.32
Presump	15	59.72	0.12	66.85	1.51	5.82
Presump	16	58.93	0.07	66.09	0.84	5.32
Presump	17	58.5	0.05	65.85	0.61	5.34
Presump	18	57.75	0.04	64.82	0.48	5.56
Presump	19	56.75	0.04	63.89	0.48	5.84
Presump	20	55.75	0.06	61.90	0.74	6.37

Existing: Q10\Scour

		Z	Froude	Water_Elev_ft	Vel_Mag_ft_p_s	
Reach	Station	Min	Ave	Ave	Ave	Max
Presump	1	70.61	0.03	78.11	0.30	1.19
Presump	2	70.17	0.02	77.98	0.29	1.24
Presump	3	69.6	0.03	77.98	0.33	1.32
Presump	4	69.32	0.02	77.91	0.25	1.34
Presump	5	68.32	0.04	76.94	0.52	5.26
Presump	6	68.02	0.07	76.57	0.73	4.58
Presump	7	67.12	0.06	74.95	0.59	6.75
Presump	8	66.2	0.08	73.39	0.87	5.18
Presump	9	65.55	0.11	72.81	1.28	5.86
Presump	10	64.55	0.10	71.92	1.14	5.46
Presump	11	63.24	0.08	70.74	0.97	4.99
Presump	12	62.01	0.10	69.50	1.17	5.41
Presump	13	61.23	0.05	68.69	0.69	5.12
Presump	14	60.54	0.05	67.90	0.64	5.32
Presump	15	59.72	0.12	66.85	1.51	5.82
Presump	16	58.93	0.07	66.09	0.84	5.32
Presump	17	58.5	0.05	65.85	0.61	5.34
Presump	18	57.75	0.04	64.82	0.48	5.56
Presump	19	56.75	0.04	63.89	0.48	5.84
Presump	20	55.75	0.06	61.90	0.74	6.37

Existing: Q25\Scour

		Z	Froude	Water_Elev_ft	Vel_Mag_ft_p_s	
Reach	Station	Min	Ave	Ave	Ave	Max
Presump	1	70.61	0.03	79.88	0.47	1.58
Presump	2	70.17	0.03	79.79	0.45	1.86
Presump	3	69.6	0.04	79.69	0.54	2.03
Presump	4	69.32	0.03	79.87	0.40	1.94
Presump	5	68.32	0.06	78.58	0.61	5.98
Presump	6	68.02	0.06	78.31	0.83	4.96
Presump	7	67.12	0.09	76.36	0.81	7.46
Presump	8	66.2	0.09	75.11	1.10	5.44
Presump	9	65.55	0.13	74.37	1.39	6.33
Presump	10	64.55	0.11	73.60	1.38	6.18
Presump	11	63.24	0.10	72.37	1.10	5.67
Presump	12	62.01	0.09	71.09	1.39	6.26
Presump	13	61.23	0.07	70.43	0.89	5.86
Presump	14	60.54	0.06	69.55	0.73	6.26
Presump	15	59.72	0.12	68.72	1.81	6.84
Presump	16	58.93	0.07	67.74	0.96	6.06
Presump	17	58.5	0.05	67.40	0.70	6.08
Presump	18	57.75	0.05	66.43	0.60	6.53
Presump	19	56.75	0.04	65.13	0.59	6.69
Presump	20	55.75	0.07	63.07	0.89	7.54

Existing: Q50\Scour

Reach	Station	Z Min	Froude Ave	Water_Elev_ft Ave	Vel_Mag_ft_p_s Ave	Vel_Mag_ft_p_s Max
Presump	1	70.61	0.05	81.13	0.62	1.88
Presump	2	70.17	0.03	81.03	0.55	2.20
Presump	3	69.6	0.04	80.90	0.62	2.44
Presump	4	69.32	0.03	81.01	0.49	2.28
Presump	5	68.32	0.05	79.75	0.72	6.43
Presump	6	68.02	0.06	79.51	0.89	5.28
Presump	7	67.12	0.12	77.29	1.11	7.34
Presump	8	66.2	0.08	76.33	1.21	5.72
Presump	9	65.55	0.14	75.60	1.73	6.36
Presump	10	64.55	0.12	74.95	1.48	6.73
Presump	11	63.24	0.11	73.77	1.35	6.14
Presump	12	62.01	0.10	72.25	1.51	6.87
Presump	13	61.23	0.07	71.65	0.96	6.36
Presump	14	60.54	0.05	70.80	0.84	6.79
Presump	15	59.72	0.13	69.87	1.98	7.44
Presump	16	58.93	0.07	68.89	1.09	6.64
Presump	17	58.5	0.10	68.48	0.83	6.57
Presump	18	57.75	0.05	67.41	0.68	7.25
Presump	19	56.75	0.05	66.06	0.65	7.37
Presump	20	55.75	0.07	63.95	1.00	8.34

Existing: Q100\Scour

Reach	Station	Z Min	Froude Ave	Water_Elev_ft Ave	Vel_Mag_ft_p_s Ave	Vel_Mag_ft_p_s Max
Presump	1	70.61	0.04	81.03	0.47	1.44
Presump	2	70.17	0.03	80.97	0.42	1.69
Presump	3	69.6	0.03	80.90	0.47	1.85
Presump	4	69.32	0.02	81.03	0.37	1.73
Presump	5	68.32	0.05	79.75	0.72	6.43
Presump	6	68.02	0.06	79.51	0.89	5.28
Presump	7	67.12	0.12	77.29	1.11	7.34
Presump	8	66.2	0.08	76.33	1.21	5.72
Presump	9	65.55	0.14	75.60	1.73	6.36
Presump	10	64.55	0.12	74.95	1.48	6.73
Presump	11	63.24	0.11	73.77	1.35	6.14
Presump	12	62.01	0.10	72.25	1.51	6.87
Presump	13	61.23	0.07	71.65	0.96	6.36
Presump	14	60.54	0.05	70.80	0.84	6.79
Presump	15	59.72	0.13	69.87	1.98	7.44
Presump	16	58.93	0.07	68.89	1.09	6.64
Presump	17	58.5	0.10	68.48	0.83	6.57
Presump	18	57.75	0.05	67.41	0.68	7.25
Presump	19	56.75	0.05	66.06	0.65	7.37
Presump	20	55.75	0.07	63.95	1.00	8.34

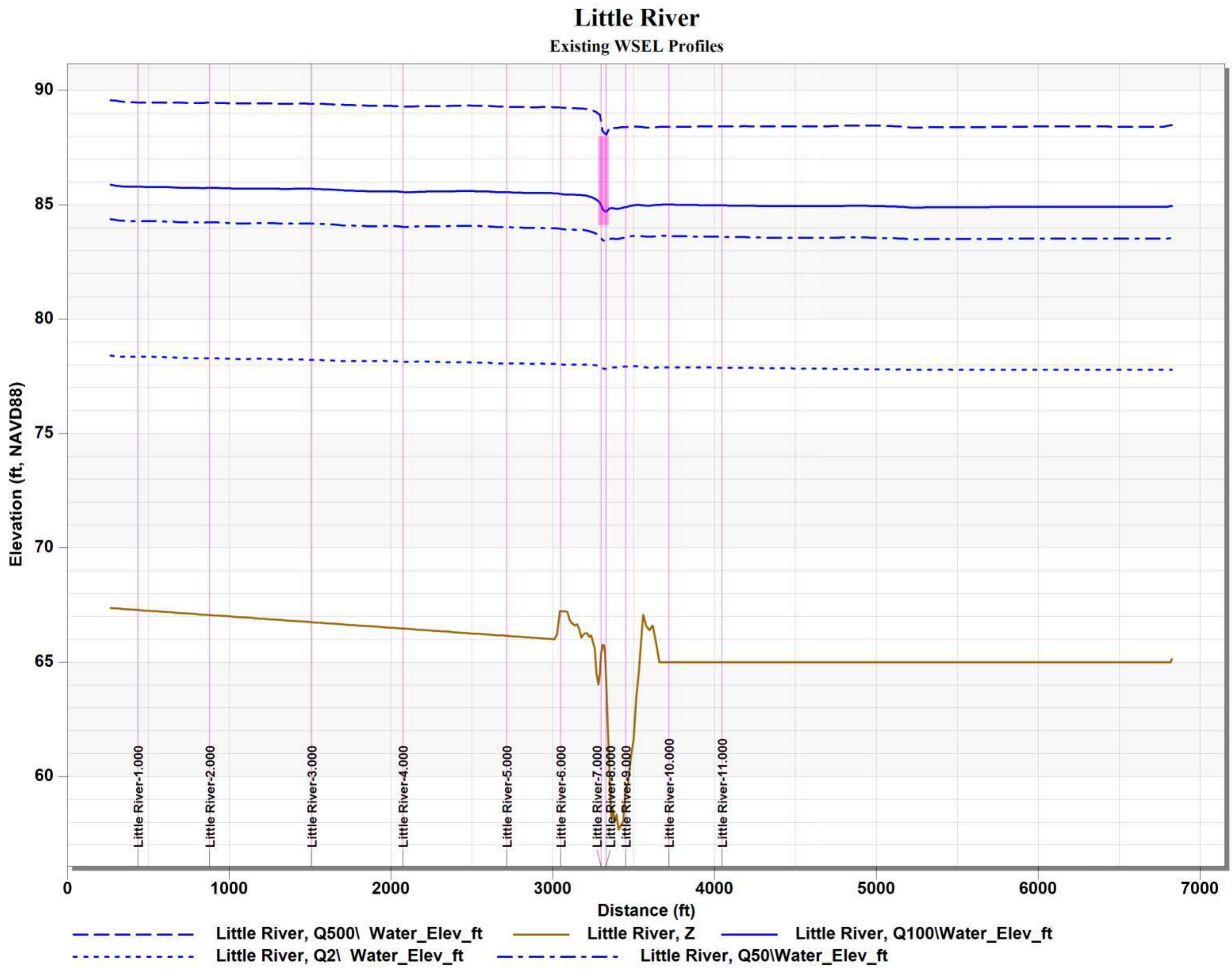
Existing: Q200\Scour

		Z	*Froude*	*Water_Elev_ft*	*Vel_Mag_ft_p_s*	
Reach	Station	Min	Ave	Ave	Ave	Max
Presump	1	70.61	0.04	82.77	0.65	1.93
Presump	2	70.17	0.03	82.62	0.58	2.23
Presump	3	69.6	0.04	82.47	0.67	2.50
Presump	4	69.32	0.03	82.51	0.49	2.37
Presump	5	68.32	0.05	81.25	0.80	7.01
Presump	6	68.02	0.06	81.00	0.97	5.71
Presump	7	67.12	0.12	78.76	1.30	6.97
Presump	8	66.2	0.10	77.90	1.35	6.24
Presump	9	65.55	0.16	77.40	2.03	6.60
Presump	10	64.55	0.16	76.39	1.89	7.23
Presump	11	63.24	0.13	75.61	1.73	6.42
Presump	12	62.01	0.15	73.87	1.88	7.61
Presump	13	61.23	0.13	73.06	1.30	6.84
Presump	14	60.54	0.08	72.52	1.02	7.47
Presump	15	59.72	0.13	71.35	2.21	8.49
Presump	16	58.93	0.09	70.59	1.27	7.34
Presump	17	58.5	0.08	70.00	1.02	7.21
Presump	18	57.75	0.06	68.94	0.77	7.97
Presump	19	56.75	0.05	67.33	0.75	8.09
Presump	20	55.75	0.10	65.11	1.10	9.57

Existing: Q500\Scour

		Z	*Froude*	*Water_Elev_ft*	*Vel_Mag_ft_p_s*	
Reach	Station	Min	Ave	Ave	Ave	Max
Presump	1	70.61	0.03	82.67	0.47	1.39
Presump	2	70.17	0.02	82.56	0.42	1.63
Presump	3	69.6	0.03	82.47	0.48	1.80
Presump	4	69.32	0.02	82.52	0.36	1.70
Presump	5	68.32	0.05	81.25	0.80	7.01
Presump	6	68.02	0.06	81.00	0.97	5.71
Presump	7	67.12	0.12	78.76	1.30	6.97
Presump	8	66.2	0.10	77.90	1.35	6.24
Presump	9	65.55	0.16	77.40	2.03	6.60
Presump	10	64.55	0.16	76.39	1.89	7.23
Presump	11	63.24	0.13	75.61	1.73	6.42
Presump	12	62.01	0.15	73.87	1.88	7.61
Presump	13	61.23	0.13	73.06	1.30	6.84
Presump	14	60.54	0.08	72.52	1.02	7.47
Presump	15	59.72	0.13	71.35	2.21	8.49
Presump	16	58.93	0.09	70.59	1.27	7.34
Presump	17	58.5	0.08	70.00	1.02	7.21
Presump	18	57.75	0.06	68.94	0.77	7.97
Presump	19	56.75	0.05	67.33	0.75	8.09
Presump	20	55.75	0.13	65.55	1.11	9.57

Little River 1D Cross-Sections


Little River
Existing WSEL Profiles
Elevation (ft, NAVD88)
90
85
80
75
70
65
60
Distance (ft)
0
1000
2000
3000
4000
5000
6000
7000
Little River-1.000
Little River-2.000
Little River-3.000
Little River-4.000
Little River-5.000
Little River-6.000
Little River-7.000
Little River-8.000
Little River-9.000
Little River-10.000
Little River-11.000
Little River, Q500\ Water_Elev_ft
Little River, Z
Little River, Q100\Water_Elev_ft
Little River, Q2\ Water_Elev_ft
Little River, Q50\Water_Elev_ft

Existing: 1996 Cal\Calibration

		Z	*Froude*	*Water_Elev_ft*	*Vel_Mag_ft_p_s*	
Reach	Station	Min	Ave	Ave	Ave	Max
Little	1	67.28	0.06	88.01	1.12	5.41
Little	2	67.06	0.05	87.95	1.02	4.89
Little	3	66.75	0.06	88.13	1.15	4.42
Little	4	66.47	0.07	87.76	1.36	4.70
Little	5	66.15	0.05	87.74	1.05	3.95
Little	6	66.91	0.09	87.75	1.32	4.09
Little	7	65.29	0.28	87.27	2.36	8.62
Little	8	64.14	0.65	86.56	3.01	9.65
Little	9	58.90	0.11	86.42	1.74	6.50
Little	10	65.00	0.06	86.74	1.20	4.26
Little	11	65.00	0.06	86.86	1.03	3.95

Existing: 2007 Cal\Calibration

		Z	*Froude*	*Water_Elev_ft*	*Vel_Mag_ft_p_s*	
Reach	Station	Min	Ave	Ave	Ave	Max
Little	1	67.28	0.03	81.88	0.36	2.03
Little	2	67.06	0.02	81.86	0.35	1.94
Little	3	66.75	0.03	81.99	0.38	1.81
Little	4	66.47	0.04	81.90	0.44	1.85
Little	5	66.15	0.02	81.81	0.33	1.65
Little	6	67.01	0.02	81.98	0.34	1.72
Little	7	64.33	0.04	81.78	0.54	2.54
Little	8	62.90	0.03	81.80	0.46	2.61
Little	9	58.94	0.02	81.79	0.35	1.60
Little	10	65.00	0.03	81.78	0.39	1.38
Little	11	65.00	0.02	81.74	0.29	1.36

Existing: Q2\Dam In Place

		Z	*Froude*	*Water_Elev_ft*	*Vel_Mag_ft_p_s*	
Reach	Station	Min	Ave	Ave	Ave	Max
Little	1	67.28	0.03	78.43	0.44	3.12
Little	2	67.06	0.05	78.29	0.50	3.07
Little	3	66.75	0.04	78.34	0.50	2.99
Little	4	66.47	0.05	78.13	0.51	3.07
Little	5	66.15	0.03	78.35	0.39	2.79
Little	6	67.01	0.04	78.03	0.46	2.91
Little	7	64.33	0.07	78.01	0.76	3.91
Little	8	62.90	0.04	77.99	0.58	4.03
Little	9	58.94	0.03	77.92	0.39	2.18
Little	10	65.00	0.03	78.08	0.43	2.28
Little	11	65.00	0.03	77.95	0.37	2.12

Existing: Q50\Dam In Place

		Z	*Froude*	*Water_Elev_ft*	*Vel_Mag_ft_p_s*	
Reach	Station	Min	Ave	Ave	Ave	Max
Little	1	67.28	0.05	84.29	0.89	4.56
Little	2	67.06	0.05	84.40	0.80	4.30
Little	3	66.75	0.06	84.17	0.92	3.92
Little	4	66.47	0.07	84.11	1.09	4.16
Little	5	66.15	0.06	84.02	0.84	3.53
Little	6	67.01	0.05	84.02	0.82	3.82
Little	7	64.33	0.12	83.70	1.47	6.11
Little	8	62.90	0.09	83.60	1.22	6.53
Little	9	58.94	0.06	83.56	1.01	4.21
Little	10	65.00	0.06	83.67	0.98	3.29
Little	11	65.00	0.04	83.66	0.75	3.21

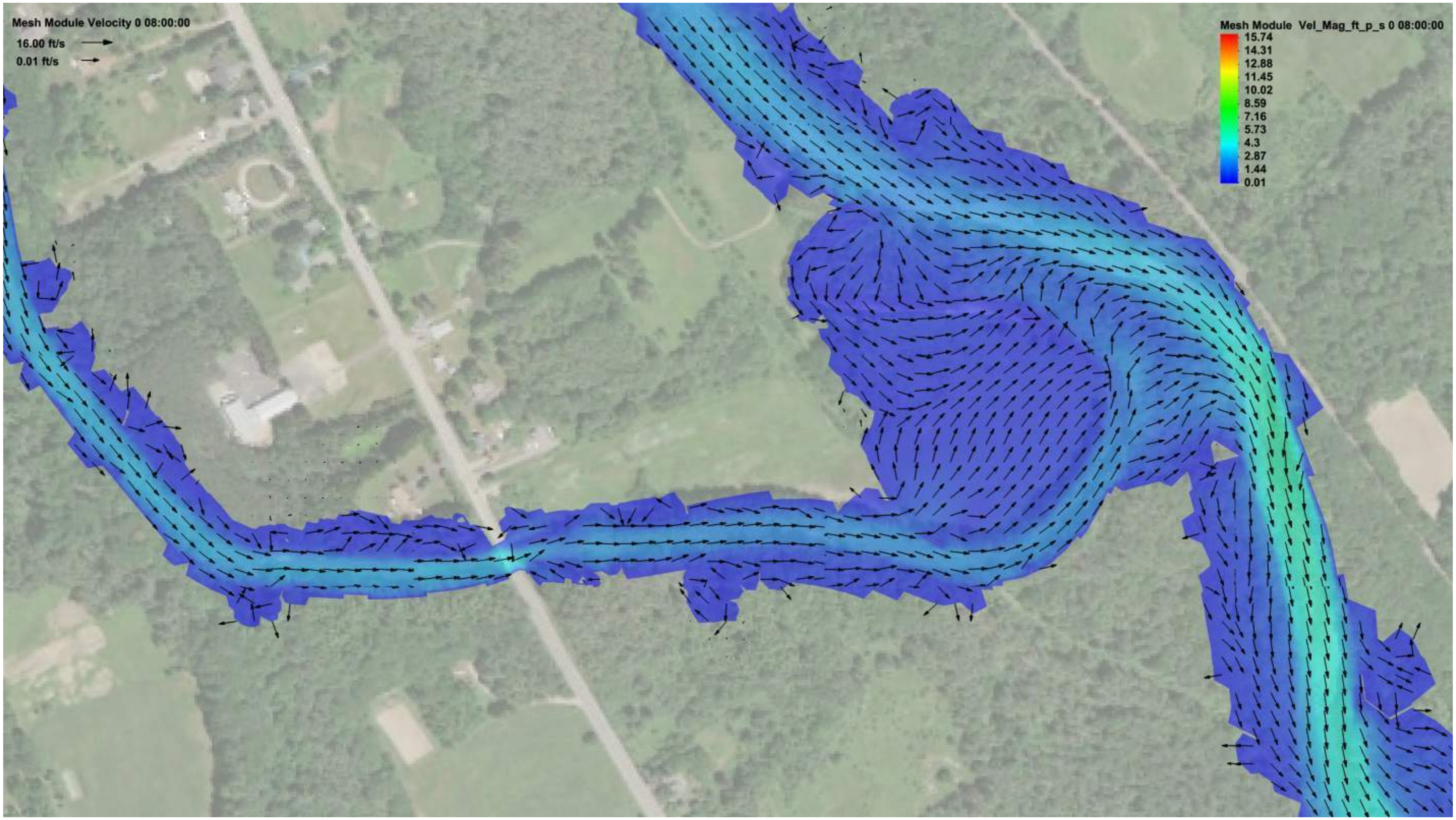
Existing: Q100\Dam In Place

		Z	*Froude*	*Water_Elev_ft*	*Vel_Mag_ft_p_s*	
Reach	Station	Min	Ave	Ave	Ave	Max
Little	1	67.28	0.05	85.79	0.94	4.72
Little	2	67.06	0.05	85.77	0.86	4.35
Little	3	66.75	0.05	85.90	0.96	3.94
Little	4	66.47	0.07	85.56	1.14	4.20
Little	5	66.15	0.05	85.56	0.88	3.54
Little	6	67.01	0.09	85.57	0.99	3.80
Little	7	64.33	0.12	85.07	1.54	6.64
Little	8	62.9	0.09	84.82	1.36	7.17
Little	9	58.94	0.07	84.88	1.15	4.73
Little	10	65.00	0.06	85.04	1.03	3.44
Little	11	65.00	0.05	84.98	0.80	3.32

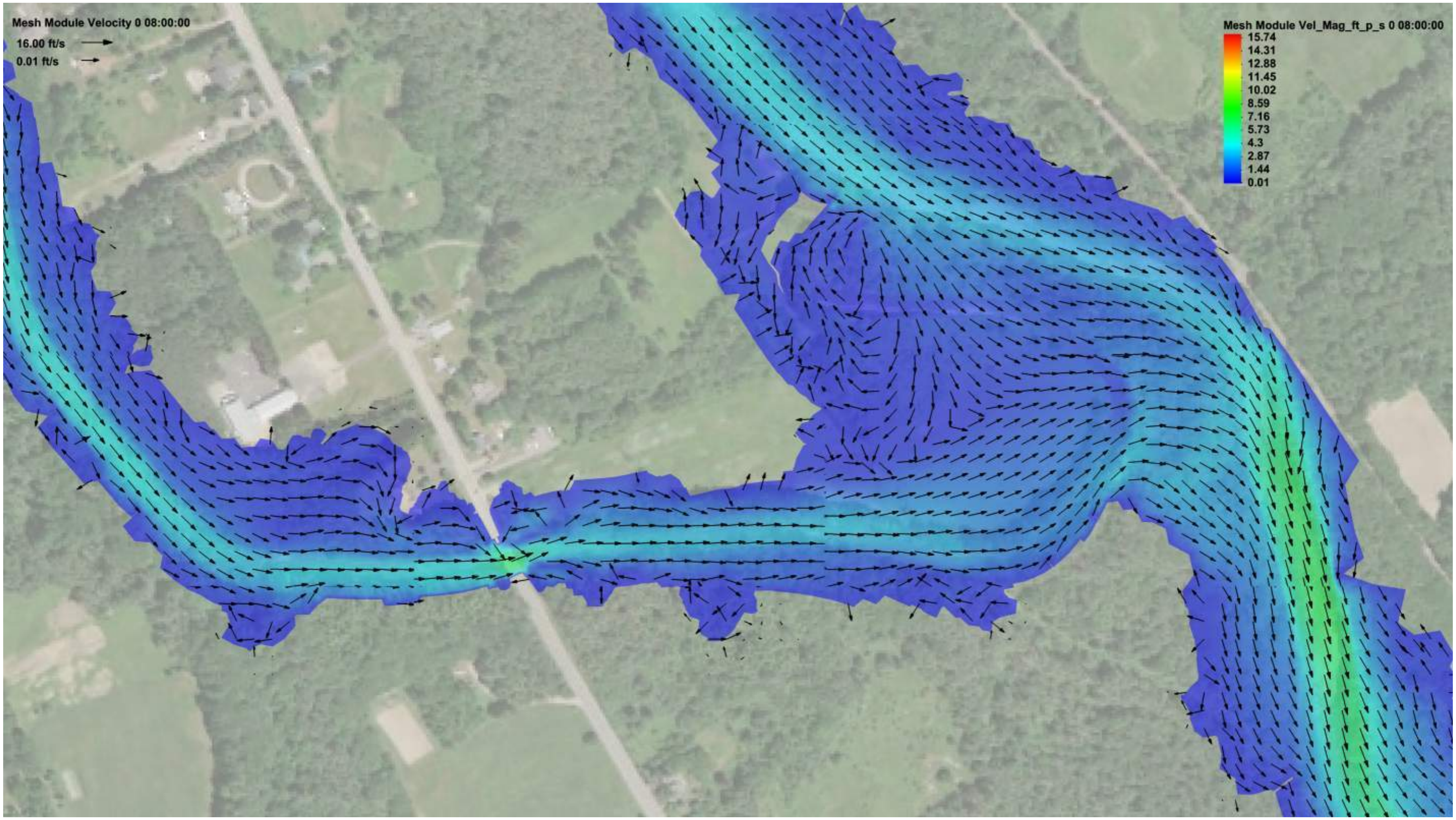
Existing: Q500\Dam In Place

		Z	*Froude*	*Water_Elev_ft*	*Vel_Mag_ft_p_s*	
Reach	Station	Min	Ave	Ave	Ave	Max
Little	1	67.28	0.05	89.48	1.05	4.91
Little	2	67.06	0.05	89.45	0.95	4.34
Little	3	66.75	0.06	89.49	1.07	3.93
Little	4	66.47	0.07	89.30	1.25	4.14
Little	5	66.15	0.05	89.30	0.96	3.48
Little	6	67.01	0.07	89.26	1.19	3.51
Little	7	64.33	0.39	89.02	3.28	7.19
Little	8	62.9	0.79	88.26	5.02	8.14
Little	9	58.94	0.08	88.39	1.33	5.19
Little	10	65.00	0.07	88.41	1.18	4.41
Little	11	65.00	0.06	88.69	0.93	3.80

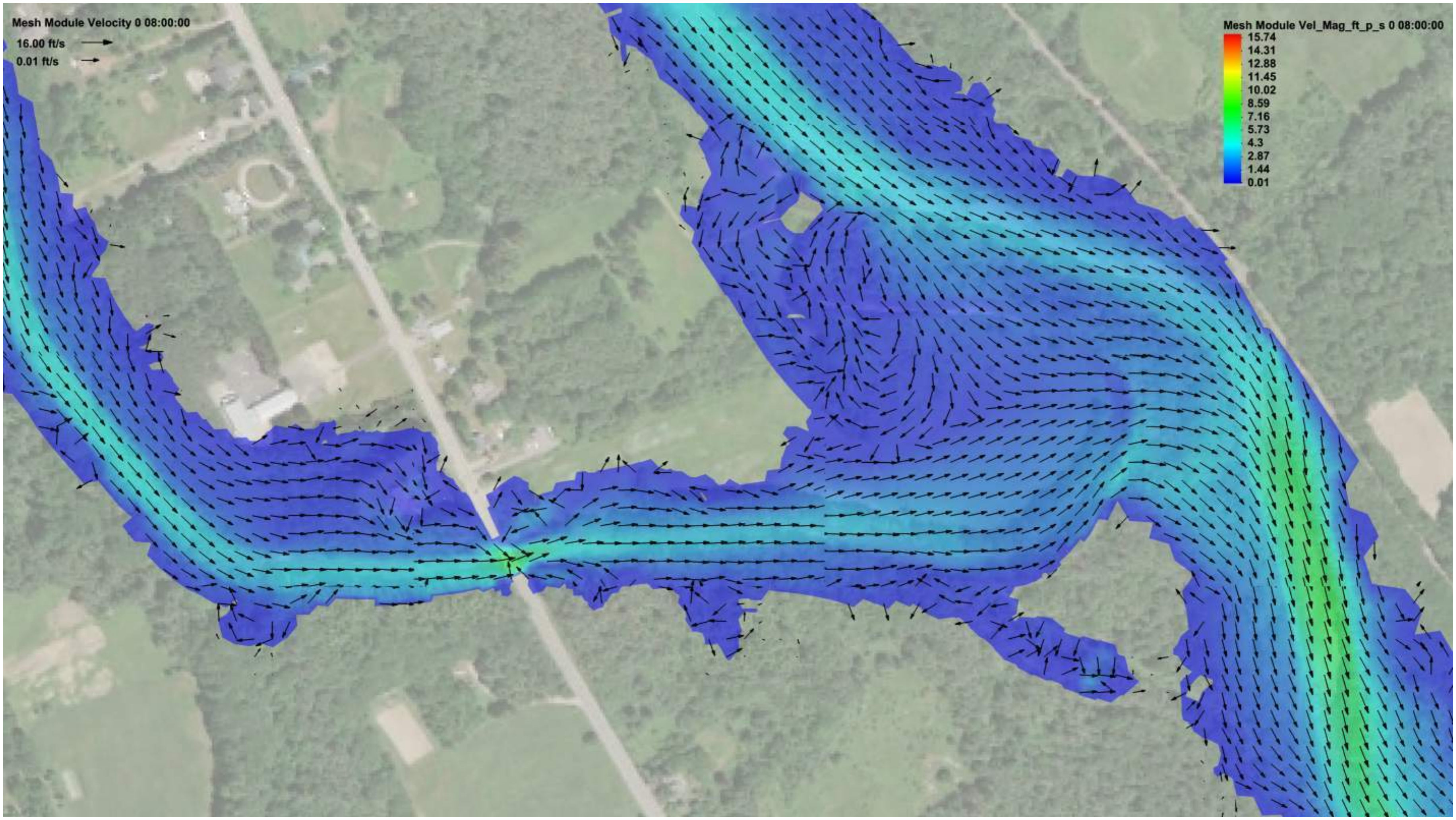
Existing Conditions: Q2 Velocity Profile



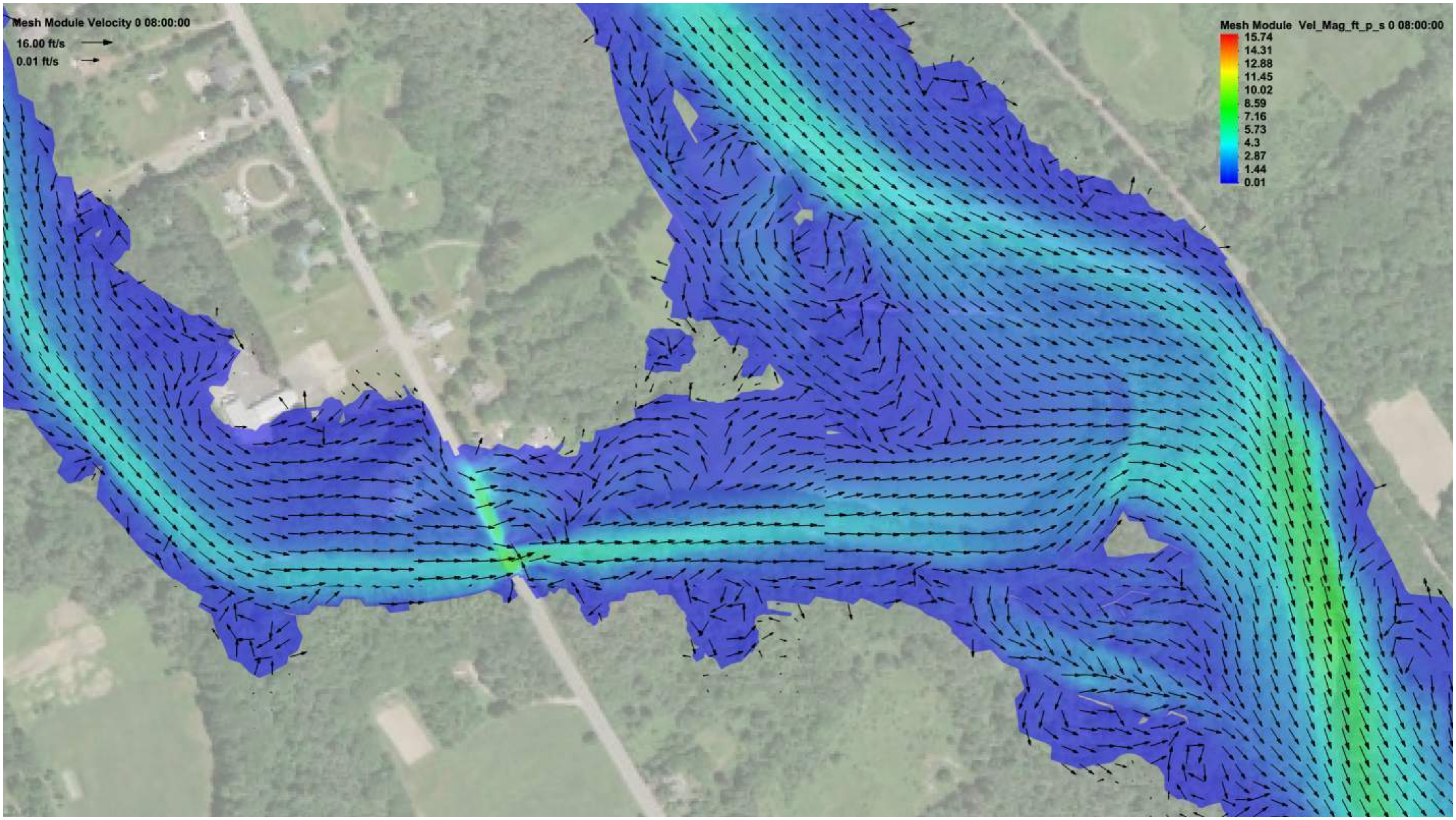
Existing Conditions: Q50 Velocity Profile

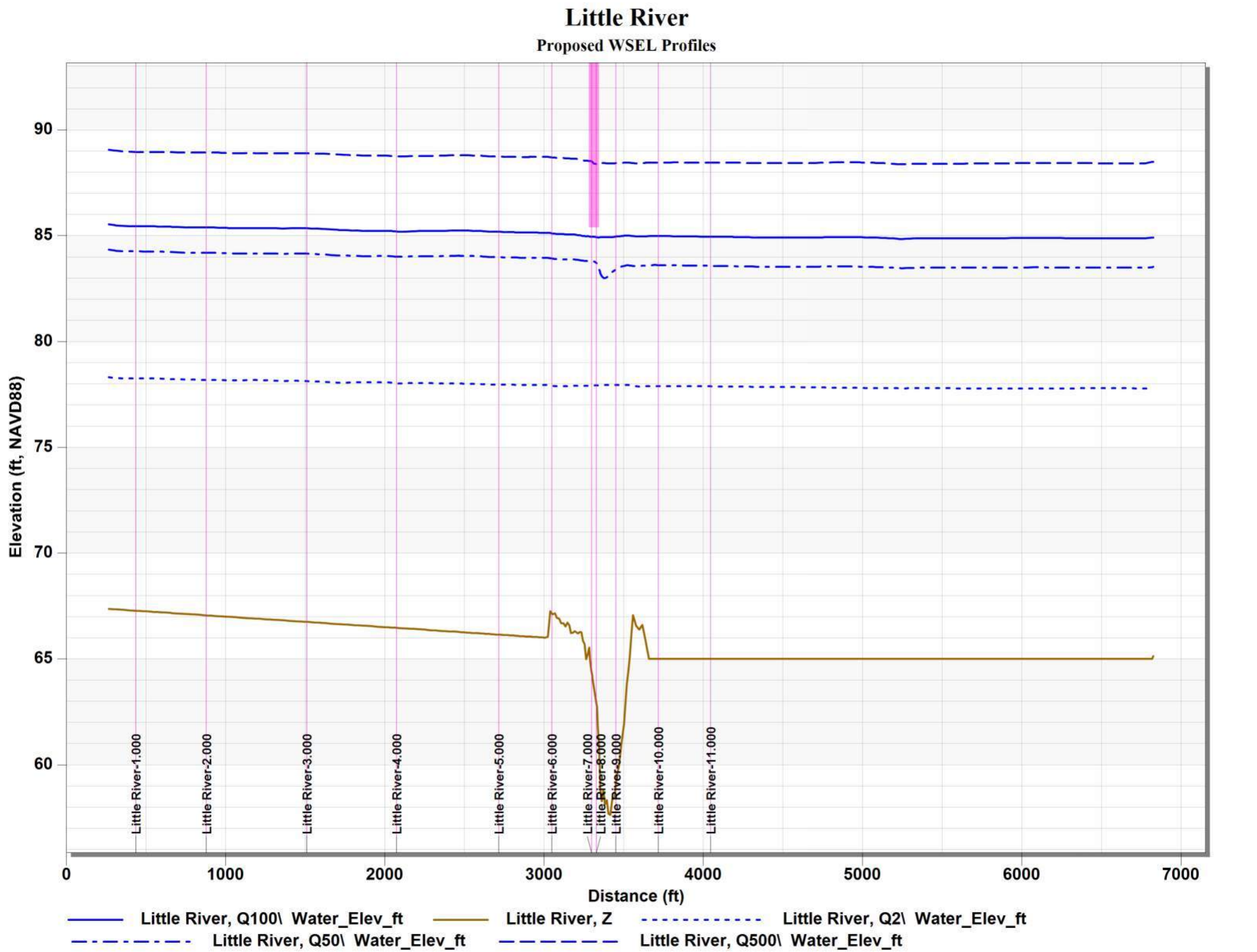


Existing Conditions: Q100 Velocity Profile



Existing Conditions: Q500 Velocity Profile





Little River
Proposed WSEL Profiles
Elevation (ft, NAVD88)
90
85
80
75
70
65
60
Distance (ft)
0
1000
2000
3000
4000
5000
6000
7000
Little River-1.000
Little River-2.000
Little River-3.000
Little River-4.000
Little River-5.000
Little River-6.000
Little River-7.000
Little River-8.000
Little River-9.000
Little River-10.000
Little River-11.000
Little River, Q100\ Water_Elev_ft
Little River, Z
Little River, Q2\ Water_Elev_ft
Little River, Q50\ Water_Elev_ft
Little River, Q500\ Water_Elev_ft

Proposed: Q2\Dam In Place

		Z	*Froude*	*Water_Elev_ft*	*Vel_Mag_ft_p_s*	
Reach	Station	Min	Ave	Ave	Ave	Max
Little	1.00	67.28	0.03	78.32	0.44	3.16
Little	2.00	67.06	0.05	78.20	0.52	3.11
Little	3.00	66.75	0.04	78.25	0.51	3.03
Little	4.00	66.47	0.05	78.03	0.51	3.11
Little	5.00	66.15	0.04	78.13	0.42	2.85
Little	6.00	67.01	0.04	77.93	0.44	3.00
Little	7.00	64.33	0.03	77.93	0.46	2.39
Little	8.00	62.90	0.02	77.93	0.44	2.25
Little	9.00	58.94	0.02	77.96	0.32	1.69
Little	10.00	65.00	0.03	78.07	0.43	2.23
Little	11.00	65.00	0.03	78.18	0.38	2.12

Proposed: Q50\Dam In Place

		Z	*Froude*	*Water_Elev_ft*	*Vel_Mag_ft_p_s*	
Reach	Station	Min	Ave	Ave	Ave	Max
Little	1.00	67.28	0.05	84.26	0.90	4.57
Little	2.00	67.06	0.05	84.52	0.85	4.33
Little	3.00	66.75	0.06	84.14	0.92	3.94
Little	4.00	66.47	0.07	84.08	1.08	4.17
Little	5.00	66.15	0.05	84.04	0.84	3.57
Little	6.00	67.01	0.05	84.02	0.82	3.83
Little	7.00	64.33	0.05	83.84	0.96	4.37
Little	8.00	62.90	0.09	83.71	1.49	5.43
Little	9.00	58.94	0.10	83.40	1.46	5.35
Little	10.00	65.00	0.06	83.62	0.97	3.31
Little	11.00	65.00	0.04	83.59	0.75	3.24

Proposed: Q100\Dam In Place

		Z	*Froude*	*Water_Elev_ft*	*Vel_Mag_ft_p_s*	
Reach	Station	Min	Ave	Ave	Ave	Max
Little	1.00	67.28	0.05	85.45	0.96	4.84
Little	2.00	67.06	0.05	85.72	0.91	4.53
Little	3.00	66.75	0.06	85.75	0.99	4.09
Little	4.00	66.47	0.07	85.22	1.17	4.34
Little	5.00	66.15	0.05	85.23	0.89	3.69
Little	6.00	67.01	0.06	85.27	0.92	3.96
Little	7.00	64.33	0.05	84.98	1.03	4.48
Little	8.00	62.90	0.05	84.93	1.02	4.68
Little	9.00	58.94	0.06	85.03	1.01	3.98
Little	10.00	65.00	0.06	85.02	1.03	3.34
Little	11.00	65.00	0.05	85.27	0.80	3.29

Proposed: Q500\Dam In Place

		Z	*Froude*	*Water_Elev_ft*	*Vel_Mag_ft_p_s*	
Reach	Station	Min	Ave	Ave	Ave	Max
Little	1	67.28	0.06	88.96	1.05	5.09
Little	2	67.06	0.06	88.91	1.01	4.56
Little	3	66.75	0.06	88.98	1.09	4.11
Little	4	66.47	0.07	88.75	1.28	4.34
Little	5	66.15	0.05	88.74	0.99	3.67
Little	6	67.01	0.07	88.72	1.22	3.74
Little	7	64.33	0.26	88.60	2.19	4.73
Little	8	62.90	0.34	88.38	2.71	5.23
Little	9	58.94	0.05	88.42	1.07	4.41
Little	10	65.00	0.06	88.46	1.14	3.63
Little	11	65.00	0.08	88.69	1.09	3.35

Proposed: Q50\No Dam

		Z	*Froude*	*Water_Elev_ft*	*Vel_Mag_ft_p_s*	
Reach	Station	Min	Ave	Ave	Ave	Max
Little	1	67.28	0.05	84.12	0.91	4.63
Little	2	67.06	0.05	84.41	0.86	4.39
Little	3	66.75	0.06	84.00	0.93	4.00
Little	4	66.47	0.07	83.95	1.09	4.23
Little	5	66.15	0.05	83.89	0.85	3.63
Little	6	67.01	0.05	83.89	0.83	3.90
Little	7	64.33	0.05	83.69	0.96	4.39
Little	8	62.90	0.09	83.57	1.44	10.47
Little	9	58.94	0.10	83.29	1.42	5.23
Little	10	65.00	0.06	83.49	0.98	3.34
Little	11	65.00	0.05	83.45	0.76	3.28

Proposed: Q100\No Dam

		Z	*Froude*	*Water_Elev_ft*	*Vel_Mag_ft_p_s*	
Reach	Station	Min	Ave	Ave	Ave	Max
Little	1	67.28	0.05	85.32	0.97	4.89
Little	2	67.06	0.05	85.40	0.92	4.59
Little	3	66.75	0.06	85.62	1.00	4.14
Little	4	66.47	0.07	85.08	1.18	4.40
Little	5	66.15	0.06	85.09	0.90	3.75
Little	6	67.01	0.06	85.01	0.91	4.02
Little	7	64.33	0.05	84.83	1.04	4.52
Little	8	62.90	0.05	84.78	1.02	4.72
Little	9	58.94	0.06	84.88	0.97	4.01
Little	10	65.00	0.06	84.88	1.04	3.38
Little	11	65.00	0.05	85.12	0.80	3.34

Proposed: Q2\Scour

		Z	*Froude*	*Water_Elev_ft*	*Vel_Mag_ft_p_s*	
Reach	Station	Min	Ave	Ave	Ave	Max
Little	1	67.28	0.04	77.11	0.50	3.79
Little	2	67.06	0.07	76.88	0.57	3.77
Little	3	66.75	0.06	77.01	0.59	3.74
Little	4	66.47	0.08	76.87	0.59	3.87
Little	5	66.15	0.04	76.48	0.49	3.59
Little	6	67.01	0.05	76.47	0.50	3.81
Little	7	64.33	0.03	76.43	0.54	2.77
Little	8	62.90	0.03	76.44	0.50	2.53
Little	9	58.94	0.02	76.47	0.34	1.83
Little	10	65.00	0.04	76.31	0.49	2.79
Little	11	65.00	0.04	76.40	0.45	2.61

Proposed: Q5\Scour

		Z	*Froude*	*Water_Elev_ft*	*Vel_Mag_ft_p_s*	
Reach	Station	Min	Ave	Ave	Ave	Max
Little	1	67.28	0.05	78.87	0.65	4.35
Little	2	67.06	0.06	78.73	0.68	4.36
Little	3	66.75	0.05	78.66	0.71	4.28
Little	4	66.47	0.07	78.59	0.83	4.31
Little	5	66.15	0.05	78.44	0.61	4.09
Little	6	67.01	0.06	78.23	0.67	4.26
Little	7	64.33	0.04	78.19	0.68	3.58
Little	8	62.90	0.04	78.19	0.67	3.38
Little	9	58.94	0.03	78.23	0.51	2.60
Little	10	65.00	0.05	78.26	0.64	3.25
Little	11	65.00	0.04	78.34	0.55	3.11

Proposed: Q10\Scour

		Z	*Froude*	*Water_Elev_ft*	*Vel_Mag_ft_p_s*	
Reach	Station	Min	Ave	Ave	Ave	Max
Little	1	67.28	0.06	79.33	0.79	5.13
Little	2	67.06	0.07	79.36	0.82	5.16
Little	3	66.75	0.07	79.12	0.85	5.13
Little	4	66.47	0.09	78.88	1.02	5.16
Little	5	66.15	0.06	78.59	0.77	4.96
Little	6	67.01	0.07	78.42	0.84	5.17
Little	7	64.33	0.05	78.37	0.84	4.44
Little	8	62.90	0.05	78.37	0.83	4.22
Little	9	58.94	0.04	78.43	0.65	3.26
Little	10	65.00	0.06	78.39	0.82	3.99
Little	11	65.00	0.05	78.48	0.71	3.86

Proposed: Q25\Scour

		Z	*Froude*	*Water_Elev_ft*	*Vel_Mag_ft_p_s*	
Reach	Station	Min	Ave	Ave	Ave	Max
Little	1	67.28	0.08	80.91	0.95	5.47
Little	2	67.06	0.07	80.74	0.94	5.41
Little	3	66.75	0.07	80.59	0.92	5.22
Little	4	66.47	0.10	80.39	1.20	5.28
Little	5	66.15	0.07	80.25	0.89	5.04
Little	6	67.01	0.07	80.26	0.94	5.19
Little	7	64.33	0.06	80.07	1.00	4.86
Little	8	62.90	0.06	80.08	0.97	4.82
Little	9	58.94	0.05	80.03	0.81	3.78
Little	10	65.00	0.08	79.95	1.03	4.05
Little	11	65.00	0.06	80.00	0.82	4.09

Proposed: Q50\Scour

		Z	*Froude*	*Water_Elev_ft*	*Vel_Mag_ft_p_s*	
Reach	Station	Min	Ave	Ave	Ave	Max
Little	1	67.28	0.08	82.04	1.04	5.67
Little	2	67.06	0.07	81.86	0.89	5.58
Little	3	66.75	0.08	81.77	1.09	5.21
Little	4	66.47	0.09	81.53	1.28	5.44
Little	5	66.15	0.06	81.50	0.96	4.96
Little	6	67.01	0.07	81.41	1.00	5.23
Little	7	64.33	0.06	81.20	1.11	5.14
Little	8	62.90	0.06	81.16	1.07	5.16
Little	9	58.94	0.05	81.22	0.91	4.12
Little	10	65.00	0.08	81.12	1.17	4.11
Little	11	65.00	0.06	81.11	0.89	4.18

Proposed: Q100\Scour

		Z	*Froude*	*Water_Elev_ft*	*Vel_Mag_ft_p_s*	
Reach	Station	Min	Ave	Ave	Ave	Max
Little	1	67.28	0.08	82.42	1.19	6.34
Little	2	67.06	0.08	82.63	1.02	6.27
Little	3	66.75	0.09	82.08	1.24	5.84
Little	4	66.47	0.11	81.84	1.49	6.17
Little	5	66.15	0.08	81.70	1.11	5.59
Little	6	67.01	0.08	81.58	1.15	5.96
Little	7	64.33	0.07	81.32	1.28	5.93
Little	8	62.90	0.07	81.28	1.24	5.97
Little	9	58.94	0.06	81.35	1.06	4.78
Little	10	65.00	0.10	81.24	1.36	4.73
Little	11	65.00	0.07	81.19	1.04	4.83

Proposed: Q200\Scour

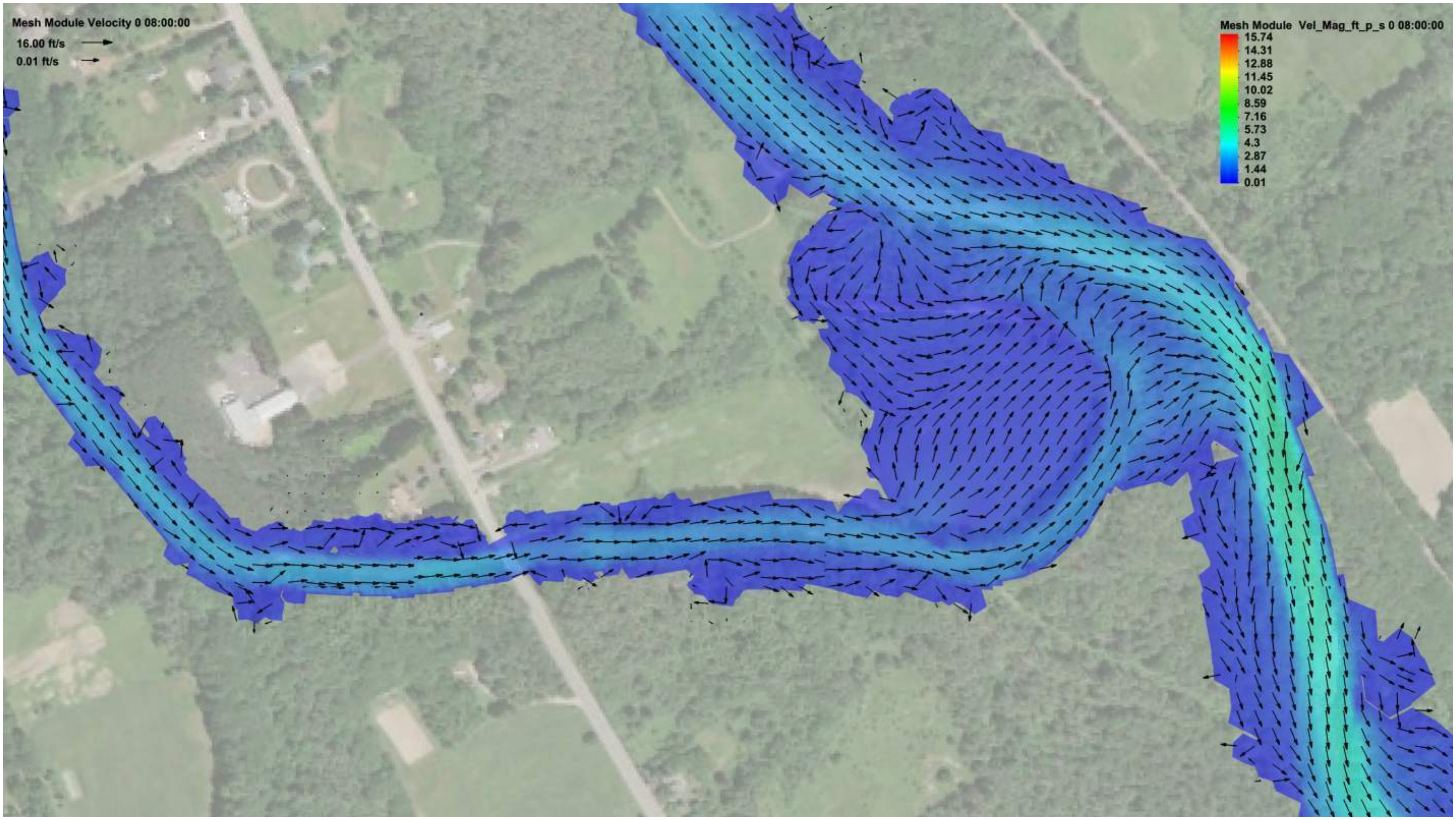
		Z	*Froude*	*Water_Elev_ft*	*Vel_Mag_ft_p_s*	
Reach	Station	Min	Ave	Ave	Ave	Max
Little	1	67.28	0.08	83.95	1.27	6.43
Little	2	67.06	0.07	84.03	1.21	6.17
Little	3	66.75	0.08	83.68	1.30	5.67
Little	4	66.47	0.11	83.52	1.56	6.07
Little	5	66.15	0.08	83.38	1.22	5.26
Little	6	67.01	0.07	83.35	1.18	5.69
Little	7	64.33	0.07	82.94	1.38	6.26
Little	8	62.90	0.10	82.80	1.68	7.14
Little	9	58.94	0.10	82.58	1.57	6.26
Little	10	65.00	0.09	82.74	1.42	4.83
Little	11	65.00	0.06	82.64	1.09	4.85

Proposed: Q500\Scour

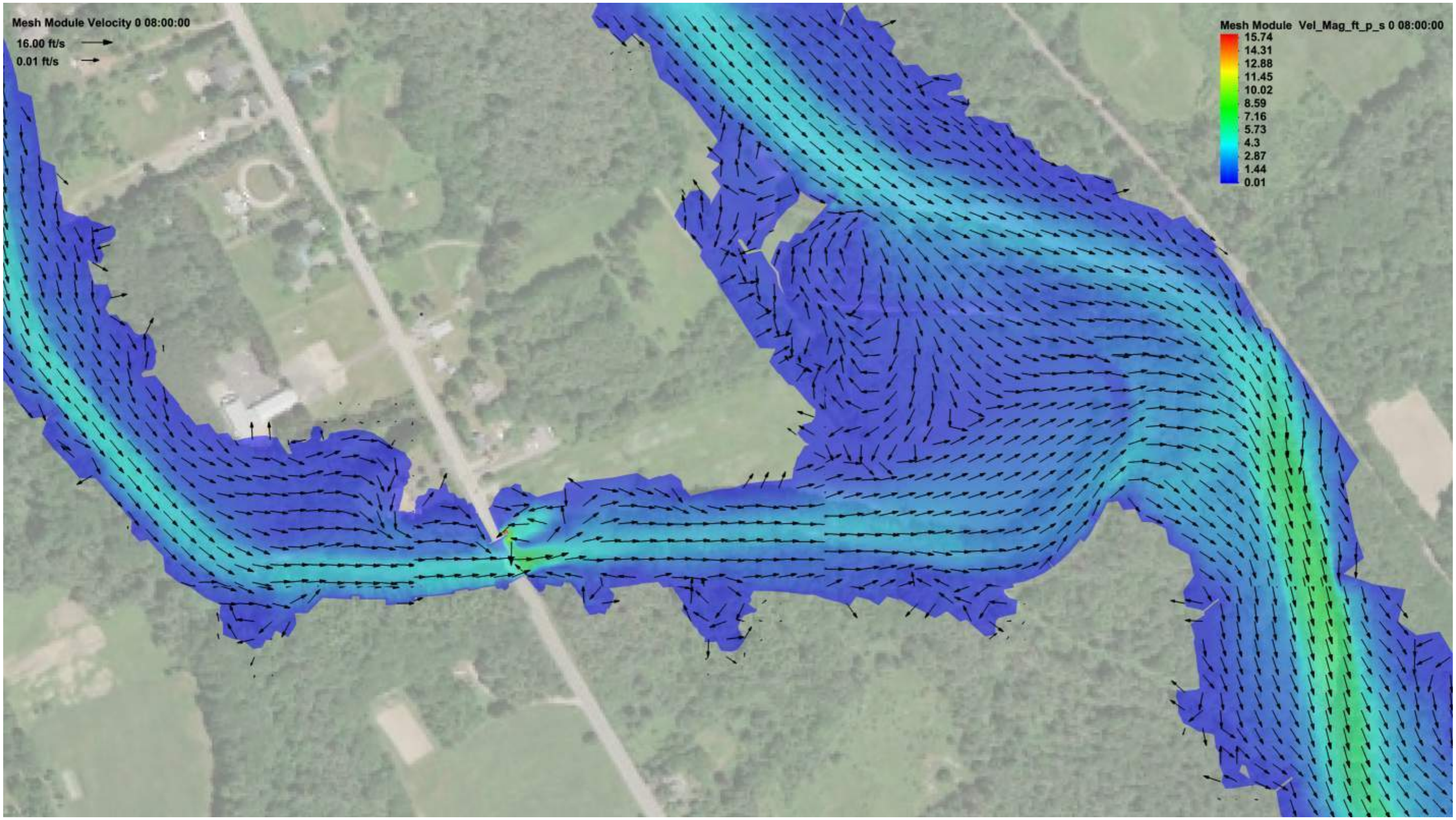
		Z	*Froude*	*Water_Elev_ft*	*Vel_Mag_ft_p_s*	
Reach	Station	Min	Ave	Ave	Ave	Max
Little	1	67.28	0.08	84.44	1.44	7.22
Little	2	67.06	0.08	84.56	1.37	6.93
Little	3	66.75	0.09	84.11	1.48	6.36
Little	4	66.47	0.12	83.84	1.78	6.86
Little	5	66.15	0.09	83.72	1.40	5.92
Little	6	67.01	0.08	83.62	1.37	6.45
Little	7	64.33	0.09	83.19	1.58	7.23
Little	8	62.90	0.11	82.96	1.88	8.16
Little	9	58.94	0.11	82.68	1.77	7.11
Little	10	65.00	0.12	82.88	1.66	5.58
Little	11	65.00	0.07	82.75	1.27	5.62

Hoyle, Tanner Project No. 923403.08
Little River Bridge, Gorham, ME
Route 237 over the Little River
MaineDOT WIN 023909.00

Proposed Conditions: Q2 Velocity Profile

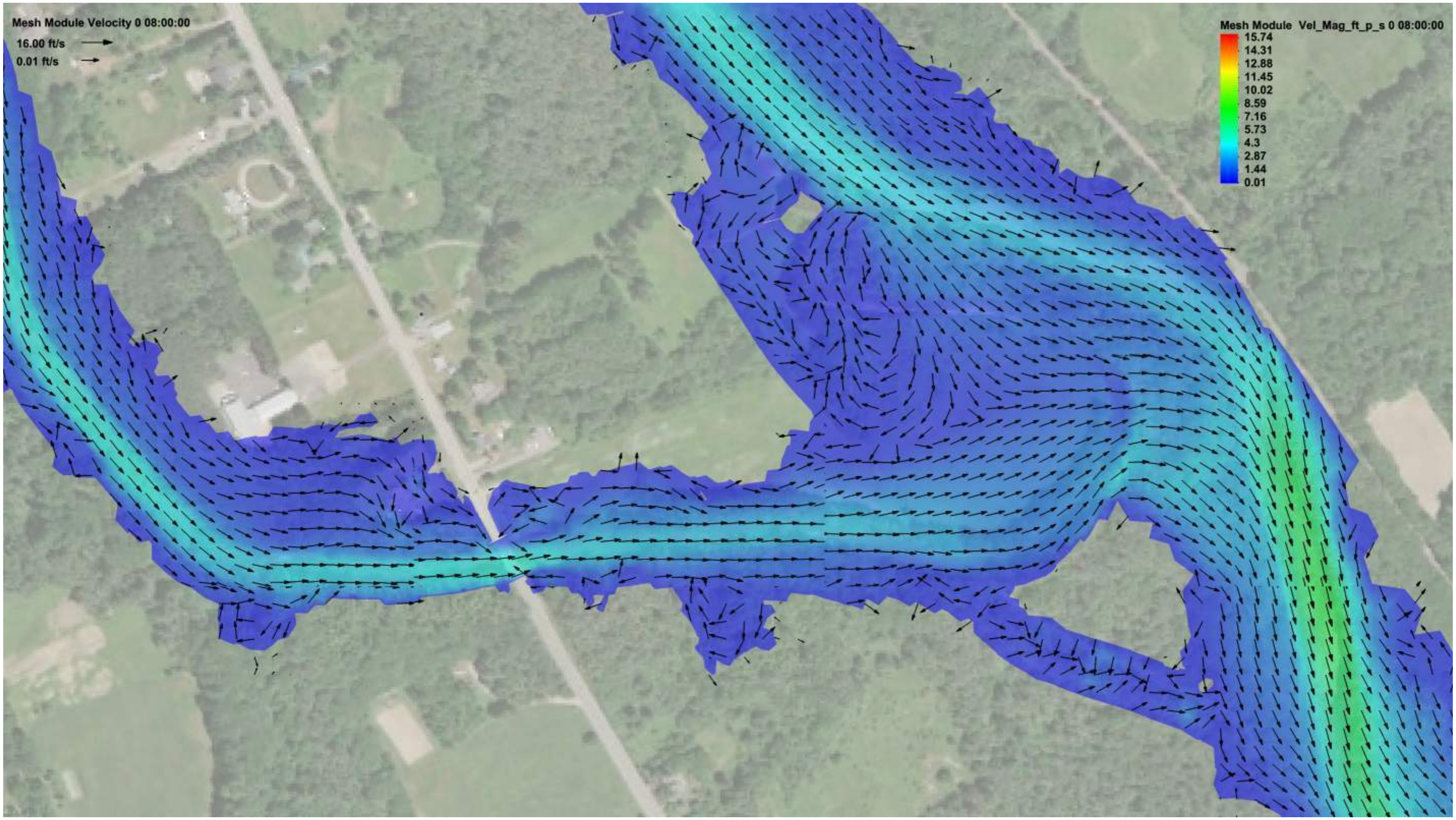


Proposed Conditions: Q50 Velocity Profile



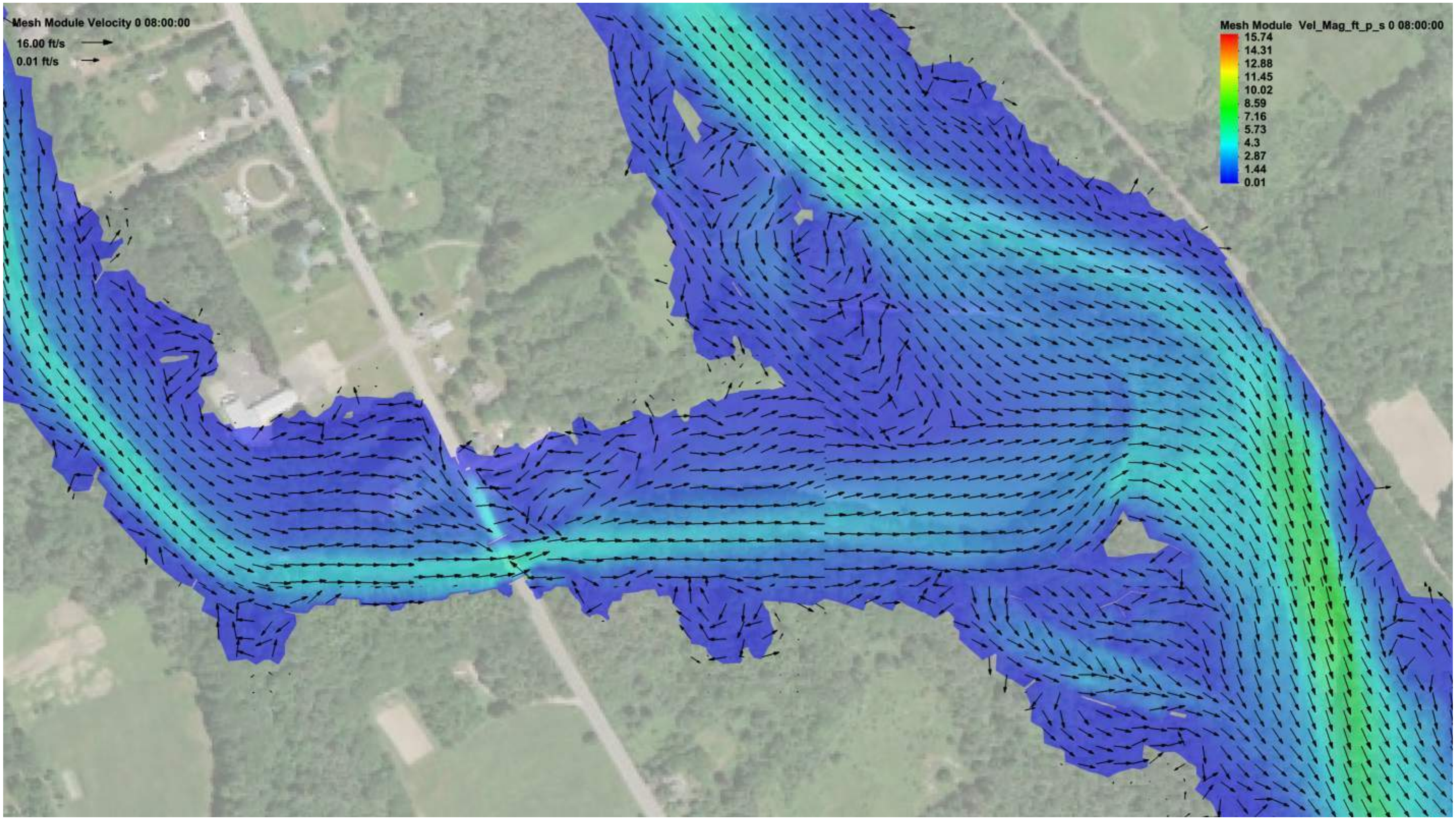
Hoyle, Tanner Project No. 923403.08
Little River Bridge, Gorham, ME
Route 237 over the Little River
MaineDOT WIN 023909.00

Proposed Conditions: Q100 Velocity Profile

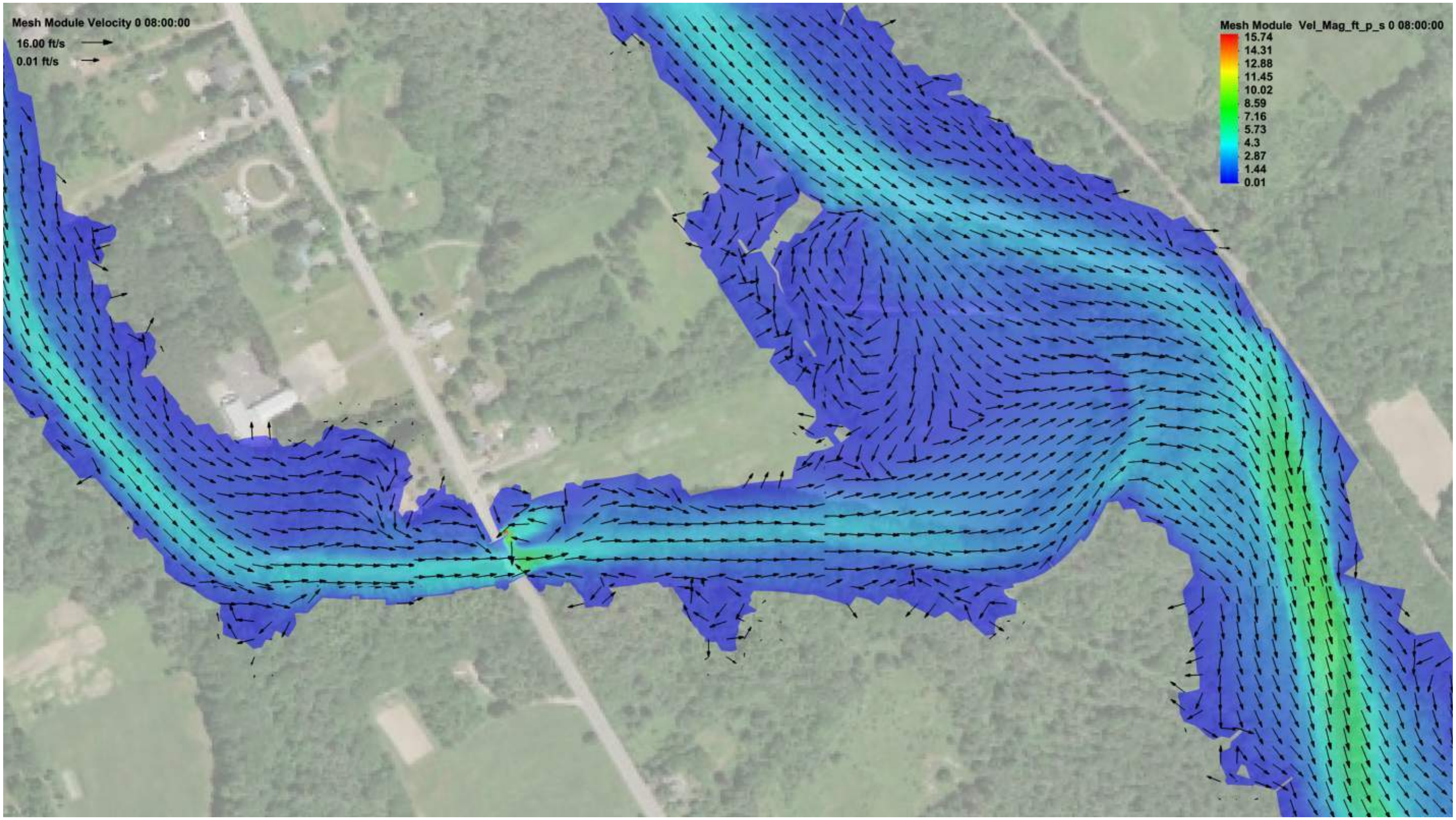


Hoyle, Tanner Project No. 923403.08
Little River Bridge, Gorham, ME
Route 237 over the Little River
MaineDOT WIN 023909.00

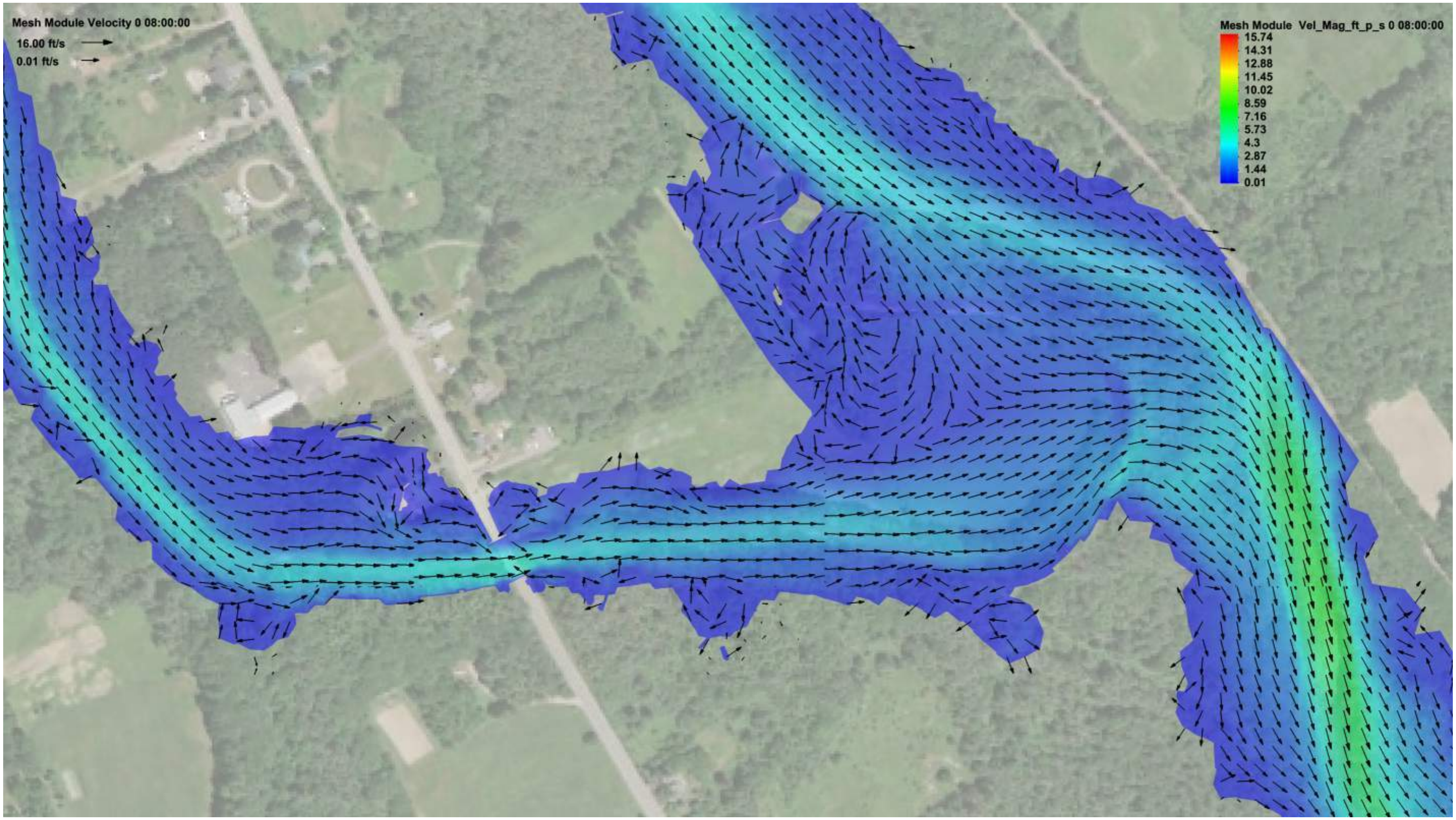
Proposed Conditions: Q500 Velocity Profile



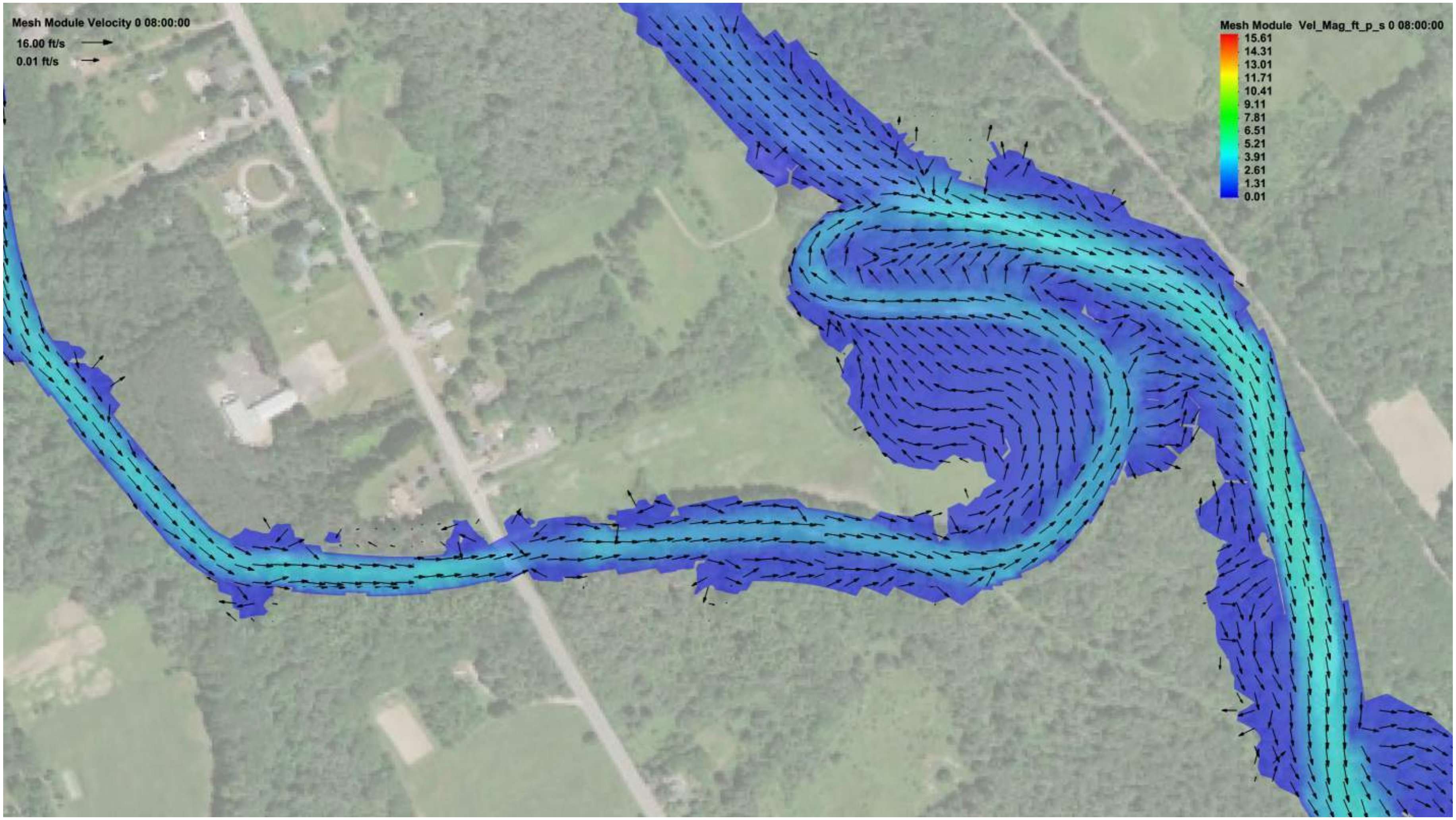
Proposed Conditions without Dam: Q50 Velocity Profile



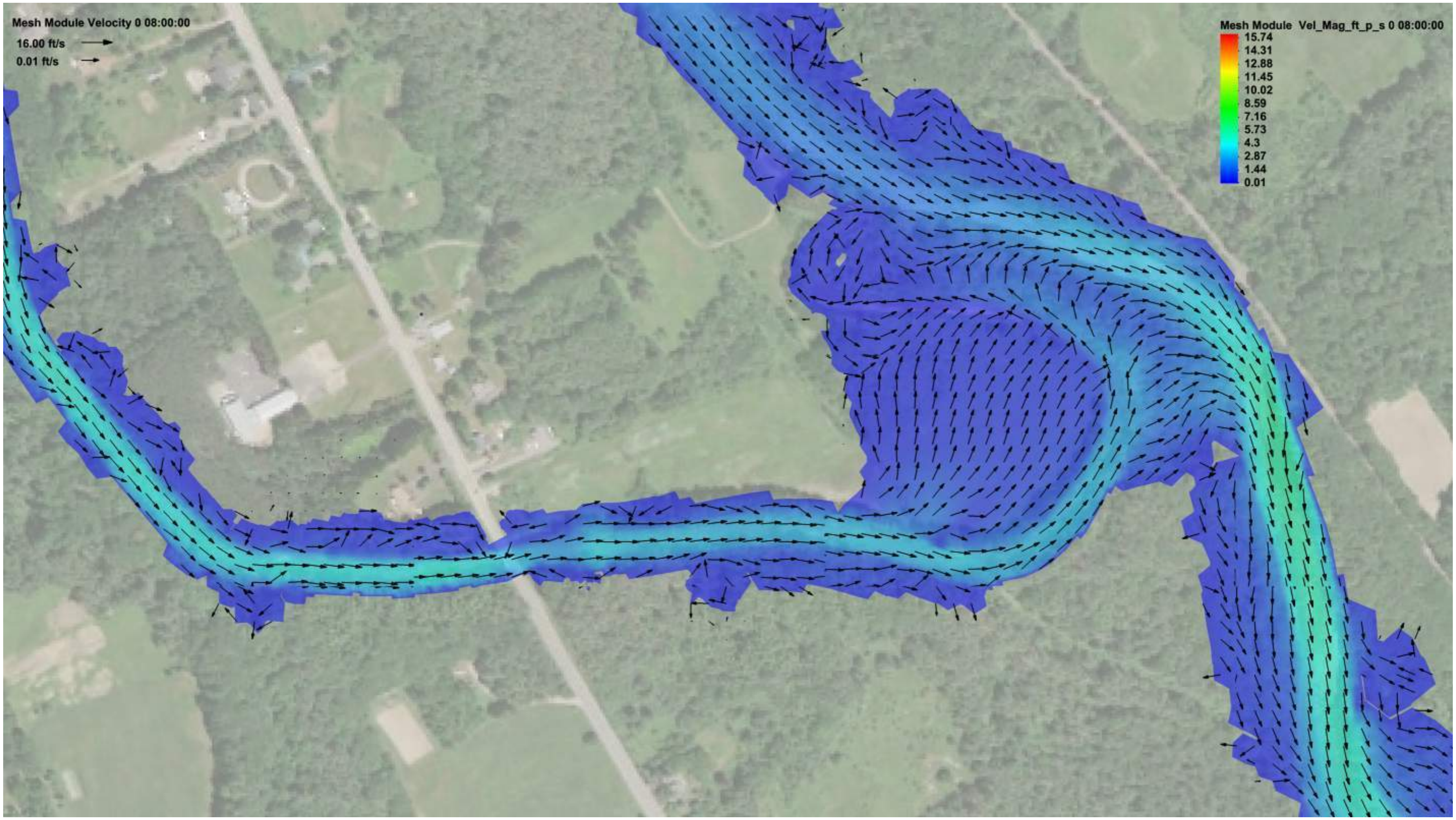
Proposed Conditions without Dam: Q100 Velocity Profile



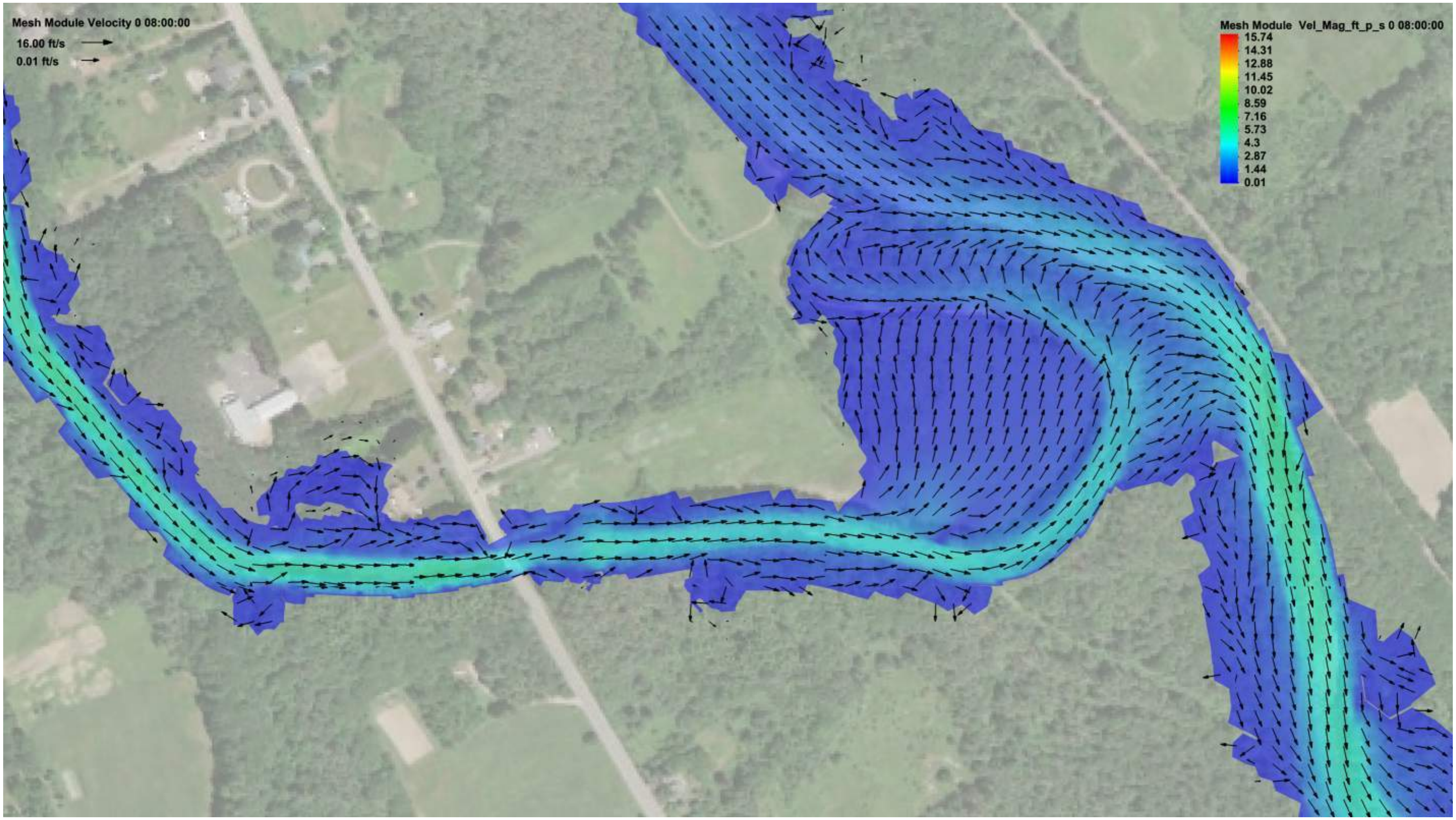
Proposed Conditions: Scour Q2 Velocity Profile



Proposed Conditions: Scour Q5 Velocity Profile

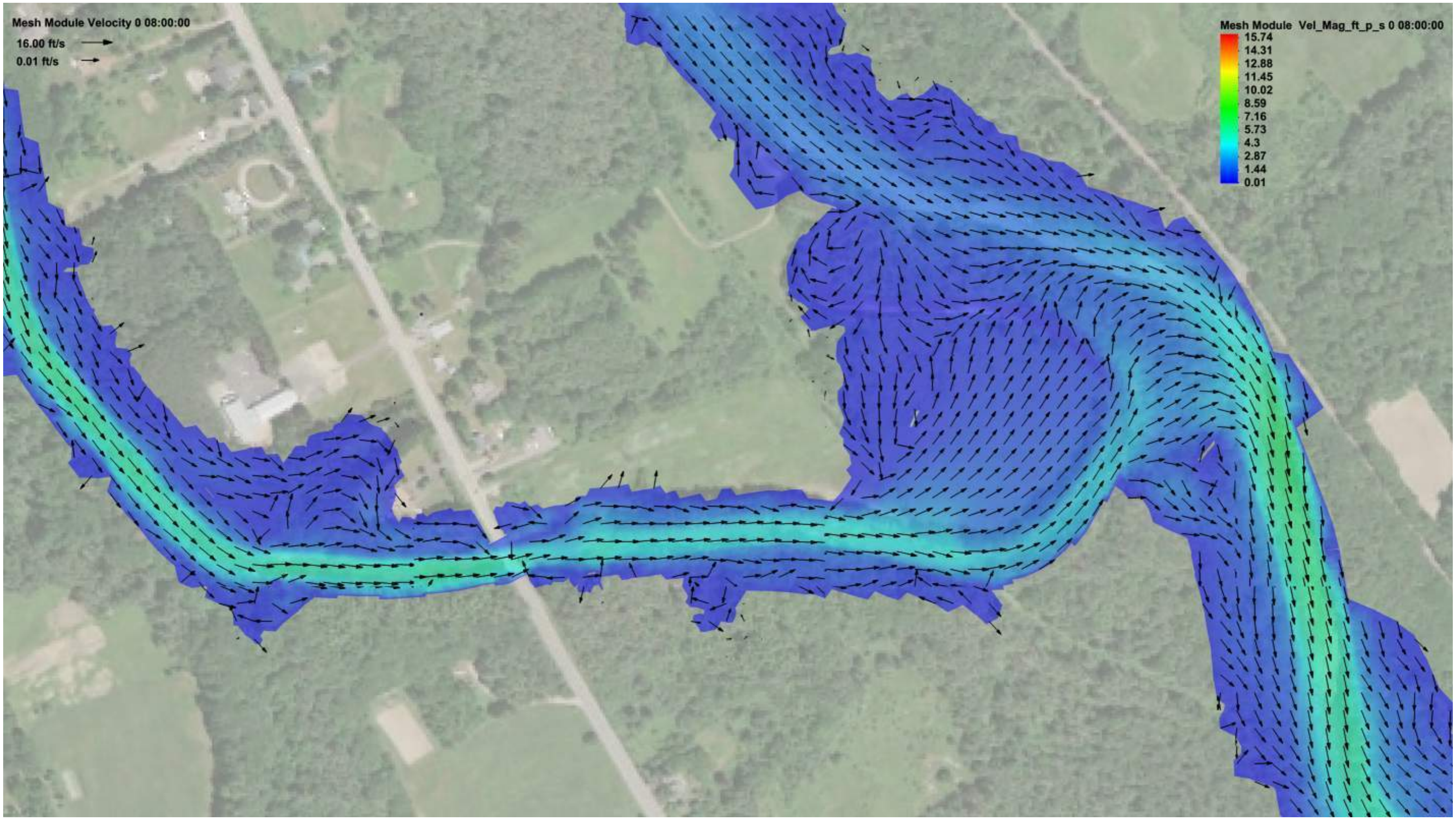


Proposed Conditions: Scour Q10 Velocity Profile

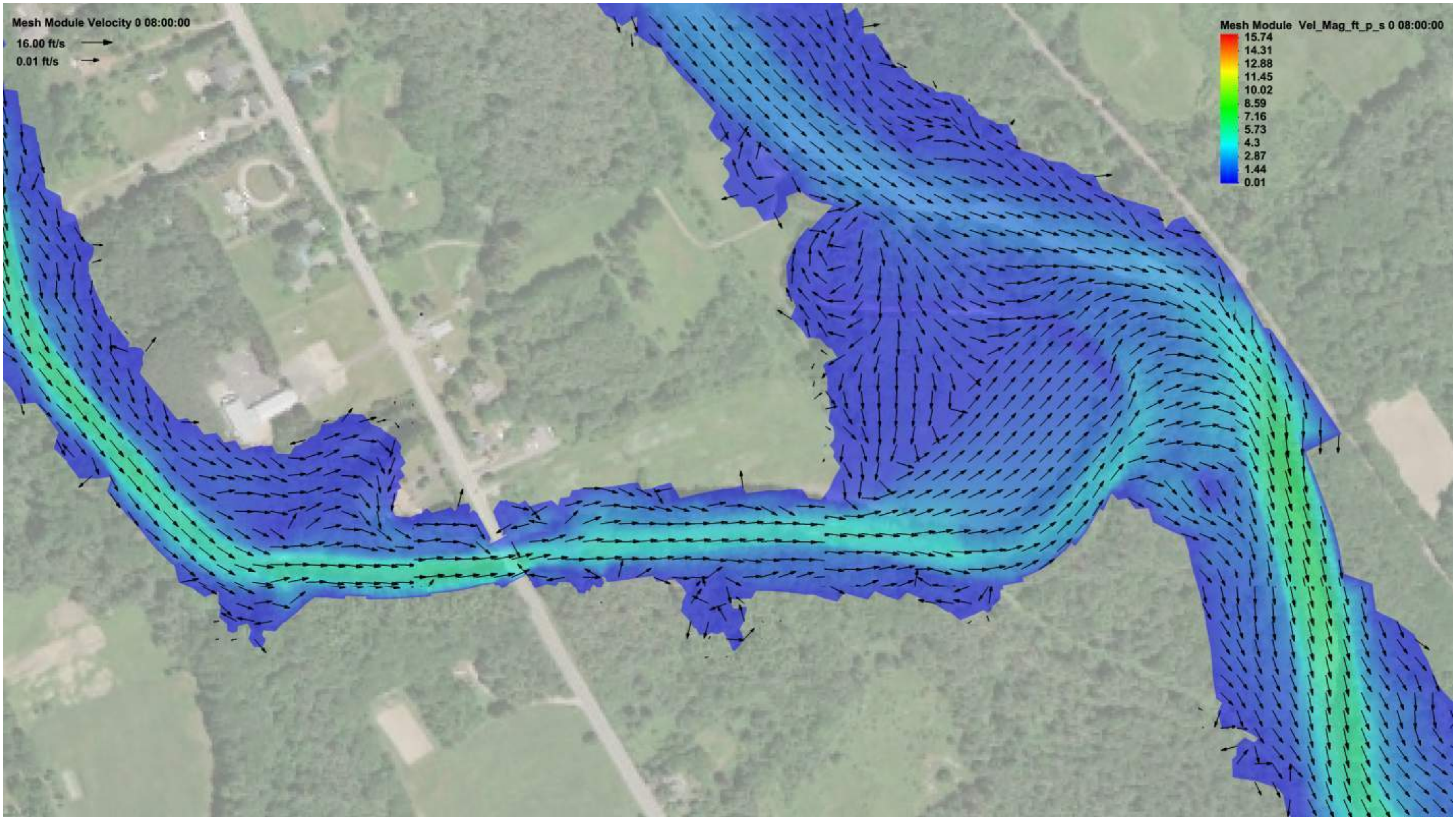


Hoyle, Tanner Project No. 923403.08
Little River Bridge, Gorham, ME
Route 237 over the Little River
MaineDOT WIN 023909.00

Proposed Conditions: Scour Q25 Velocity Profile

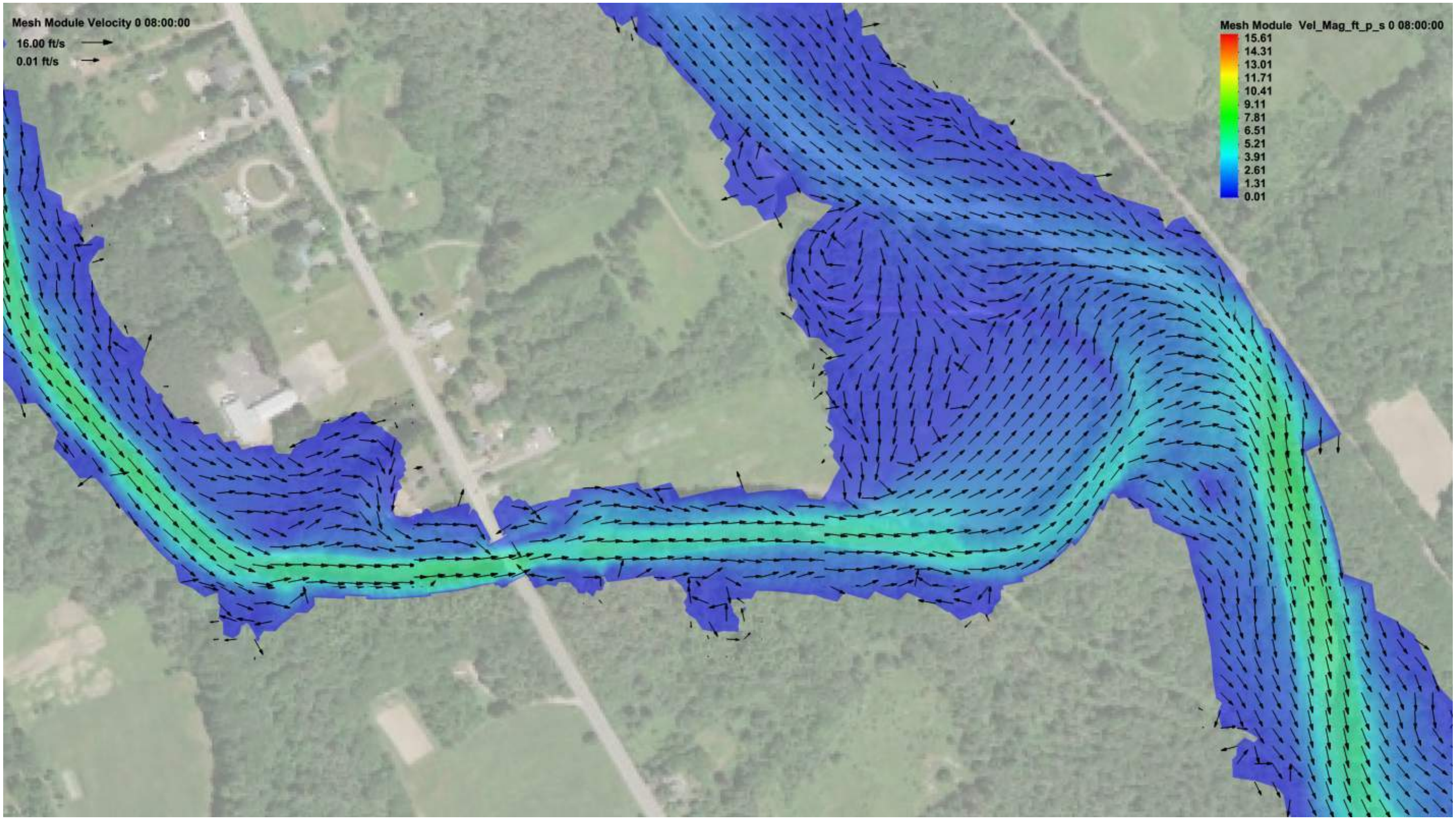


Proposed Conditions: Scour Q50 Velocity Profile

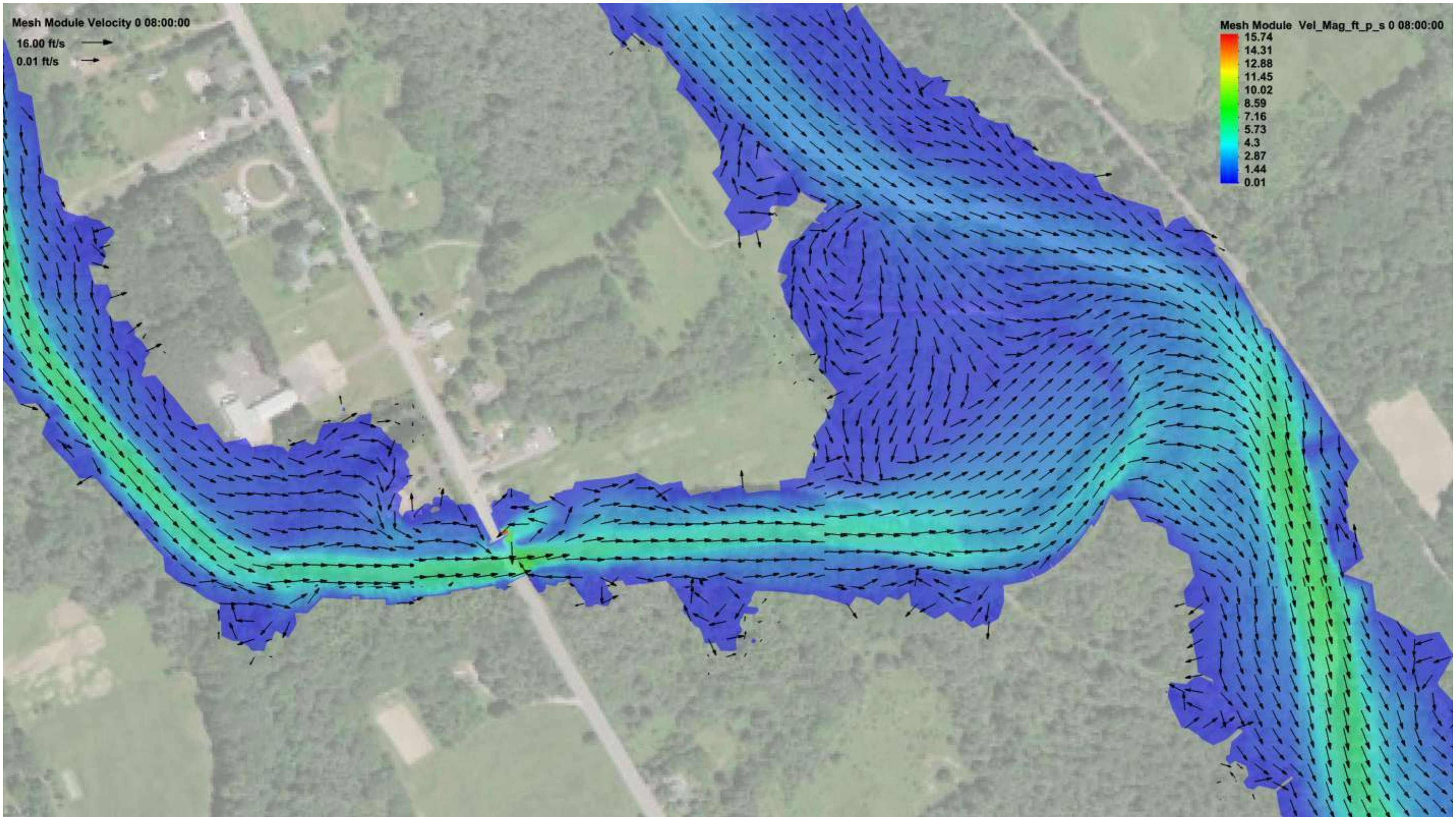


Hoyle, Tanner Project No. 923403.08
Little River Bridge, Gorham, ME
Route 237 over the Little River
MaineDOT WIN 023909.00

Proposed Conditions: Scour Q100 Velocity Profile

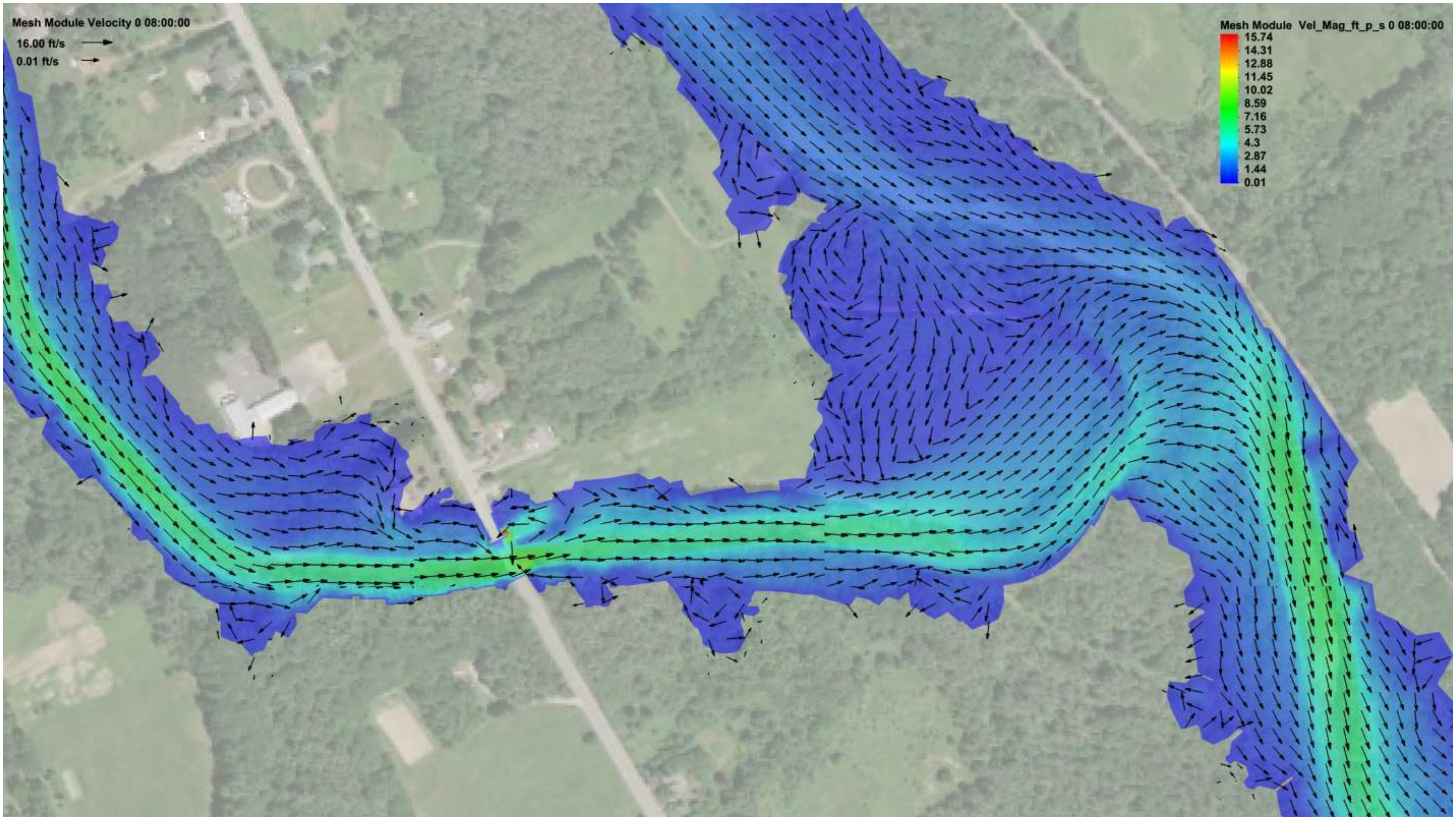


Proposed Conditions: Scour Q200 Velocity Profile



Hoyle, Tanner Project No. 923403.08
Little River Bridge, Gorham, ME
Route 237 over the Little River
MaineDOT WIN 023909.00

Proposed Conditions: Scour Q500 Velocity Profile



NOTES AND ASSUMPTIONS

D50 = 0.03 mm

References:

1. FHWA HEC 18, 5th Edition, Publication No. FHWA-HIF-12-003
2. MaineDOT Bridge Design Guide 2003 with updates to June 2018
3. Preliminary Geotechnical Letter Report prepared by Thielsch Engineering, Inc.
4. Water Resources Engineering, David A. Chin, Second Edition

- Scour is to be analyzed per FHWA Hydraulic Engineering Circular (HEC) 18.
- Proposed hydraulic data including flood velocity and elevations are taken from Proposed SRH-2D Model.
- The proposed bridge's hydraulic opening was designed for a 50 Year Flood Frequency. Per the MaineDOT Bridge Design Guide the 100 Year Flood is suggested for scour design and the 500 Year Flood is a check.
- Based on the lab results of the material from the borings taken by GZA, the bed material consists of marine clay. Therefore, clear water scour will occur. A sieve analysis was not performed on the bed material. It has been assumed that the HEC-18 Figure 6.9 will be used to estimate the grain size for the bed material. Figure 6.9 shows a transition between clay and silt at approximately 0.03 mm. For this analysis, 0.03 mm will be used to estimate critical shear stress.

GIVENS

D50 = 0.03 mm

Median Grain Size: $D_{50} := 0.03\ mm$

Conversion to Customary Units: $K_u := 1.486$

Density of Water: $\rho_w := 1.94\ \frac{lbf \cdot s^2}{ft^4}$ (62.4 pcf)

Manning's n of Channel: $n_{man} := 0.022 \cdot \frac{s}{ft^{\frac{1}{3}}}$

Gravity: $g := 32.2\ \frac{ft}{s^2}$

Calculated Shear Stress: $\tau_c := 0.05 \left(\frac{N}{m^2}\right) \cdot \left(\frac{D_{50}}{mm}\right)^{-0.4} = 0.004\ \frac{lbf}{ft^2}$ Figure 6.9, Reference 1

(FHWA reccommends using the lower bound to compute shear stress)

Specific Gravity of Riprap: $S_s := 2.65$

Figure 6.9, Reference 1

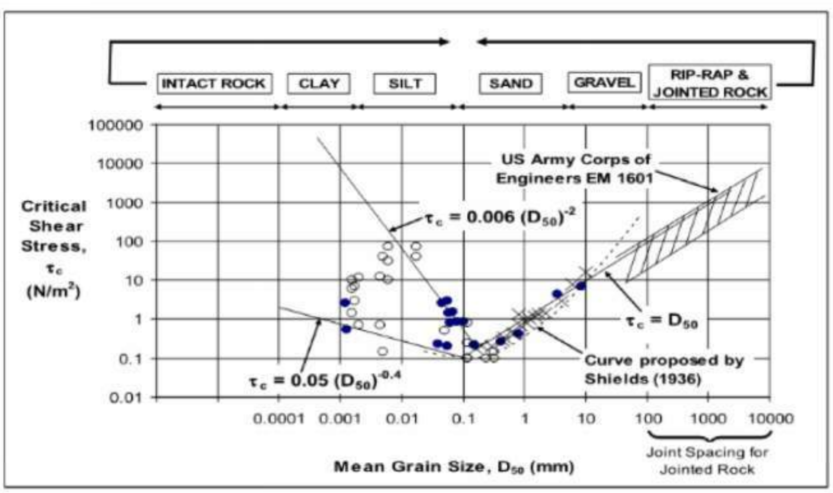


Figure 6.9. Critical shear stress versus particle size (Briaud et al. 2011).

D50 = 0.03 mm

HYDRAULIC DATA
Bridge Scour Variables

Storm Events being Analyzed:

$$Storm := \begin{bmatrix} 2 \\ 5 \\ 10 \\ 25 \\ 50 \\ 100 \\ 200 \\ 500 \end{bmatrix}$$

Max Velocity within 2 Flow Depths of Countermeasure Toe:
(flow depth is approximately the same as h)

$$V := \begin{bmatrix} 2.51 \\ 3.35 \\ 4.16 \\ 4.64 \\ 4.93 \\ 5.69 \\ 6.52 \\ 7.53 \end{bmatrix} \cdot \frac{ft}{s}$$

SMS Node 4274

Max Depth within 2 Flow Depths of Countermeasure Toe:

$$h := \begin{bmatrix} 13.70 \\ 15.49 \\ 15.66 \\ 17.27 \\ 18.42 \\ 18.54 \\ 20.02 \\ 20.15 \end{bmatrix} \cdot ft$$

SMS Node 4274

Max Unit Discharge within 2 Flow Depths of Countermeasure Toe:

$$q := \begin{array}{l} \text{for } i \in 1..\text{rows}(V) \\ \quad q_i \leftarrow V_i \cdot h_i \\ q \end{array} = \begin{bmatrix} 34.4 \\ 51.9 \\ 65.1 \\ 80.1 \\ 90.8 \\ 105.5 \\ 130.5 \\ 151.7 \end{bmatrix} \frac{ft^2}{s}$$

Shear Stress within 2 Flow Depths of Countermeasure Toe:

$$\tau_{initial} := \begin{array}{l} \text{for } i \in 1..\text{rows}(V) \\ \quad \tau_i \leftarrow \dfrac{\rho_w \cdot g \cdot \left(n_{man} \cdot V_i\right)^2}{K_u^{\,2} \cdot h_i^{\frac{1}{3}}} \\ \tau \end{array} = \begin{bmatrix} 0.036 \\ 0.062 \\ 0.095 \\ 0.114 \\ 0.126 \\ 0.167 \\ 0.214 \\ 0.285 \end{bmatrix} \frac{lbf}{ft^2}$$

Ref. 1, Eq. 6.7

D50 = 0.03 mm

TOTAL SCOUR
NCHRP 24-20 Abutment Scour Approach
Clear Water Scour:

Flow Depth Including Live-Bed Scour: $y_c := \left(\frac{\rho_w \cdot g}{\tau_c}\right)^{\frac{3}{7}} \cdot \left(\frac{n_{man} \cdot q}{K_u}\right)^{\frac{6}{7}} = \begin{bmatrix} 34.3 \\ 48.8 \\ 59.2 \\ 70.8 \\ 78.8 \\ 89.6 \\ 107.5 \\ 122.3 \end{bmatrix} ft$ Ref. 1, Eq. 8.7

Scour Amplification Factor for Clear-Water Conditions: $\alpha_{B.clear} := 1.13$ Ref. 1, Fig. 8.11 (As seen Below)
(flow within floodplain is much smaller than flow within bridge. Therefore q2f/qf approaches infinity.)

Maximum Flow Depth: $y_{max.clear} := \alpha_{B.clear} \cdot y_c = \begin{bmatrix} 38.7 \\ 55.1 \\ 67.0 \\ 80.0 \\ 89.0 \\ 101.2 \\ 121.5 \\ 138.2 \end{bmatrix} ft$ Ref. 1, Eq. 8.3

Abutment Scour Depth: $y_{s.NCHRP.clear} := y_{max.clear} - h$ Ref. 1, Eq. 8.4

$y_{s.NCHRP.clear} = \begin{bmatrix} 25.0 \\ 39.6 \\ 51.3 \\ 62.7 \\ 70.6 \\ 82.7 \\ 101.5 \\ 118.0 \end{bmatrix} ft$

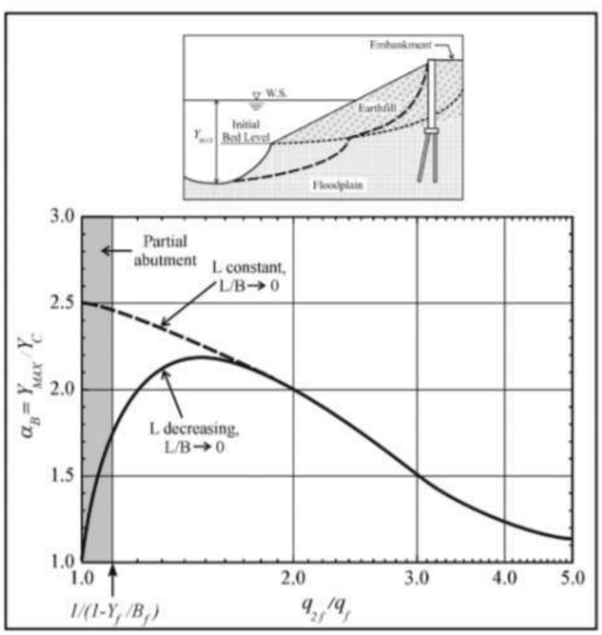


Figure 8.11. Scour amplification factor for spill-through abutments and clear-water conditions (NCHRP 2010b).

*****The existing bridge is not scour critical. Scour analysis is to be refined to account Rate of Scour with respect to life time of bridge.**

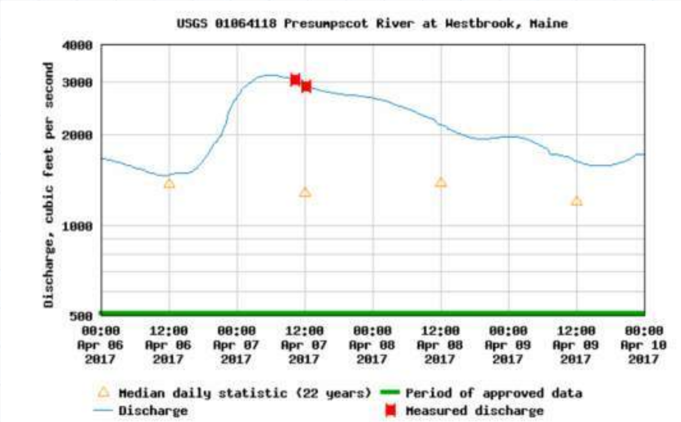
D50 = 0.03 mm

TOTAL SCOUR (CONT.)
Bridge Design Variables:

Bridge Design Life, yr: $Design_{life} := 75$

Duration of Hydrograph Peak Discharge: $Peak_{time} := 18\ hr$

Assumed based on limited USGS 01064118 Gage Data. Low flow peaks last approximately 12 hrs. Use 18 hours to account for transitional flows, ex. 280-yr flow



Cumulative Binomial Distribution: This is used to determine the probability and number of storms of interest the bridge experiences during design life. Iterate values untill cumulative probability, P = 0.9999 or less. Storm Event 50 through 500 cumulative probability may be less than 0.9999. Therefore, iterate values such that the next integer does not increase the probability.

Storm Event Occurrence within Design Life:

$$Total_{designlife} := \begin{bmatrix} 50 \\ 25 \\ 14 \\ 12 \\ 6 \\ 1 \\ 1 \\ 1 \end{bmatrix} \qquad Storm = \begin{bmatrix} 2.0 \\ 5.0 \\ 10.0 \\ 25.0 \\ 50.0 \\ 100.0 \\ 200.0 \\ 500.0 \end{bmatrix}$$

TOTAL SCOUR (CONT.)
Cumulative Binomial Distribution, Cont.:

D50 = 0.03 mm

$$
\begin{aligned}
P := & \text{for } j \in 1..\text{rows}(Storm) \\
& \quad m \leftarrow Total_{designlife_j} \\
& \quad p \leftarrow \frac{1}{Storm_j} \\
& \quad n \leftarrow Design_{life} \\
& \quad P_{old} \leftarrow 0 \\
& \quad \text{for } k \in 1..m \\
& \quad\quad P_{new} \leftarrow \frac{n!}{k! \cdot (n-k)!} \cdot p^k \cdot (1-p)^{(n-k)} \\
& \quad\quad P_{old} \leftarrow P_{new} + P_{old} \\
& \quad\quad P_j \leftarrow P_{old} \\
& \quad\quad P
\end{aligned}
= \begin{bmatrix} 0.9988 \\ 0.9979 \\ 0.9928 \\ 0.9532 \\ 0.7795 \\ 0.3565 \\ 0.2588 \\ 0.1293 \end{bmatrix}
$$

Ref. 4, Eqs. 4.4, 4.32

Total Storm Duration within Design Life:

$$Total_{storm\_peak} := Peak_{time} \cdot Total_{designlife} = \begin{bmatrix} 900.0 \\ 450.0 \\ 252.0 \\ 216.0 \\ 108.0 \\ 18.0 \\ 18.0 \\ 18.0 \end{bmatrix} hr$$

D50 = 0.03 mm

TOTAL SCOUR (CONT.)

Erodibility:

Assume that marine clay is on the border of High to Medium Erodibility Line.
Reference 1, Figure 6.11

$$Z := \begin{bmatrix} 1.308\ Pa & 0.1\ \frac{mm}{hr} \\ 40.128\ Pa & 1000\ \frac{mm}{hr} \end{bmatrix}$$

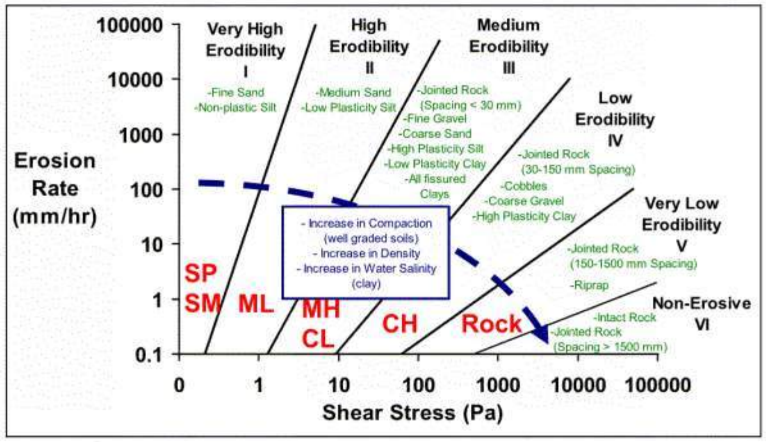


Figure 6.11. Generalized relationships for scour in cohesive materials (Briaud et al. 2011).

Calculate Erodibility, zi

Use Logarithmic interpolation for Erosion Rates listed above to determine Erosion Rate for a specific Shear Stress.

Log-Log Slope:

$$m := \frac{\log\left(\frac{Z_{1,2}}{Z_{2,2}}\right)}{\log\left(\frac{Z_{1,1}}{Z_{2,1}}\right)} = 2.7$$

Log-Log Intercept:

$$b := \log\left(\frac{Z_{2,2} \cdot \frac{hr}{mm}}{\left(\frac{Z_{2,1}}{Pa}\right)^m}\right) = -1.3$$

Erosion Rate:

$$z_i := \left(\left(\frac{\tau_{initial}}{Pa}\right)^m \cdot 10^b\right) \cdot \frac{mm}{hr} = \begin{bmatrix} 0.001 \\ 0.003 \\ 0.009 \\ 0.015 \\ 0.020 \\ 0.043 \\ 0.084 \\ 0.181 \end{bmatrix} \frac{ft}{hr} \qquad z_i = \begin{bmatrix} 0.21 \\ 0.89 \\ 2.84 \\ 4.67 \\ 6.11 \\ 13.14 \\ 25.52 \\ 55.08 \end{bmatrix} \frac{mm}{hr} \quad \text{for} \quad \tau_{initial} = \begin{bmatrix} 1.73 \\ 2.95 \\ 4.53 \\ 5.46 \\ 6.03 \\ 8.02 \\ 10.26 \\ 13.66 \end{bmatrix} Pa$$

TOTAL SCOUR (CONT.)
Rate of Scour:

D50 = 0.03 mm

Calculate Scour Depth, yt
Ref. 1, Eqs. 6.8 & 6.9, Figure 6.12

$$
\begin{aligned}
y_t :=\ & \text{for } i \in 1..\text{rows}(\tau_{initial}) \\
& \quad \text{if } i = 1 \wedge \tau_{initial_i} > \tau_c \\
& \qquad y_{t_i} \leftarrow \frac{Total_{storm\_peak_i}}{\dfrac{1}{z_{i_i}} + \dfrac{Total_{storm\_peak_i}}{y_{s.NCHRP.clear_i}}} \\
& \qquad y_t \\
& \quad \text{else if } i \geq 2 \wedge \tau_{initial_i} > \tau_{initial_{i-1}} \\
& \qquad T \leftarrow \frac{y_{s.NCHRP.clear_i} \cdot y_{t_{i-1}}}{z_{i_i} \cdot \left(y_{s.NCHRP.clear_i} - y_{t_{i-1}}\right)} + Total_{storm\_peak_i} \\
& \qquad y_{t_i} \leftarrow \frac{T}{\dfrac{1}{z_{i_i}} + \dfrac{T}{y_{s.NCHRP.clear_i}}} \\
& \qquad y_t \\
& \quad \text{else} \\
& \qquad y_{t_i} \leftarrow y_{t_{i-1}}
\end{aligned}
= \begin{bmatrix} 0.60758 \\ 1.84512 \\ 3.93161 \\ 6.70378 \\ 8.42971 \\ 9.05039 \\ 10.28409 \\ 12.92813 \end{bmatrix} ft
$$

SUMMARY

D50 = 0.03 mm

Clear Water Total Scour

Max Local Scour:

$$y_{s.NCHRP.clear} = \begin{bmatrix} 25.0 \\ 39.6 \\ 51.3 \\ 62.7 \\ 70.6 \\ 82.7 \\ 101.5 \\ 118.0 \end{bmatrix} ft \qquad Storm = \begin{bmatrix} 2.0 \\ 5.0 \\ 10.0 \\ 25.0 \\ 50.0 \\ 100.0 \\ 200.0 \\ 500.0 \end{bmatrix}$$

Local Scour within Design Life of Bridge:

$$y_t = \begin{bmatrix} 0.6 \\ 1.8 \\ 3.9 \\ 6.7 \\ 8.4 \\ 9.1 \\ 10.3 \\ 12.9 \end{bmatrix} ft \qquad Storm = \begin{bmatrix} 2.0 \\ 5.0 \\ 10.0 \\ 25.0 \\ 50.0 \\ 100.0 \\ 200.0 \\ 500.0 \end{bmatrix}$$

Scour Depth Elevation

Channel Thawleg Elevation: $El_{Thawleg} := 62.50\ ft$

Scour Elevation: $Scour_{el} := El_{Thawleg} - y_t$

$$Scour_{el} = \begin{bmatrix} 61.9 \\ 60.7 \\ 58.6 \\ 55.8 \\ 54.1 \\ 53.4 \\ 52.2 \\ 49.6 \end{bmatrix} ft \qquad Storm = \begin{bmatrix} 2.0 \\ 5.0 \\ 10.0 \\ 25.0 \\ 50.0 \\ 100.0 \\ 200.0 \\ 500.0 \end{bmatrix}$$

NOTES AND ASSUMPTIONS

D50 = 0.003 mm

References:

1. FHWA HEC 18, 5th Edition, Publication No. FHWA-HIF-12-003
2. MaineDOT Bridge Design Guide 2003 with updates to June 2018
3. Preliminary Geotechnical Letter Report prepared by Thielsch Engineering, Inc.
4. Water Resources Engineering, David A. Chin, Second Edition

- Scour is to be analyzed per FHWA Hydraulic Engineering Circular (HEC) 18.
- Proposed hydraulic data including flood velocity and elevations are taken from Proposed SRH-2D Model.
- The proposed bridge's hydraulic opening was designed for a 50 Year Flood Frequency. Per the MaineDOT Bridge Design Guide the 100 Year Flood is suggested for scour design and the 500 Year Flood is a check.
- Based on the lab results of the material from the borings taken by GZA, the bed material consists of marine clay. Therefore, clear water scour will occur. A sieve analysis was not performed on the bed material. It has been assumed that the HEC-18 Figure 6.9 will be used to estimate the grain size for the bed material. Figure 6.9 shows a transition between clay and silt at approximately 0.003 mm. For this analysis, 0.003 mm will be used to estimate critical shear stress.

D50 = 0.003 mm

GIVENS

Median Grain Size: $D_{50} := 0.003\ mm$

Conversion to Customary Units: $K_u := 1.486$

Density of Water: $\rho_w := 1.94\ \frac{lbf \cdot s^2}{ft^4}$ (62.4 pcf)

Manning's n of Channel: $n_{man} := 0.022 \cdot \frac{s}{ft^{\frac{1}{3}}}$

Gravity: $g := 32.2\ \frac{ft}{s^2}$

Calculated Shear Stress: $\tau_c := 0.05 \left(\frac{N}{m^2}\right) \cdot \left(\frac{D_{50}}{mm}\right)^{-0.4} = 0.011\ \frac{lbf}{ft^2}$ Figure 6.9, Reference 1

(FHWA reccommends using the lower bound to compute shear stress)

Specific Gravity of Riprap: $S_s := 2.65$

Figure 6.9, Reference 1

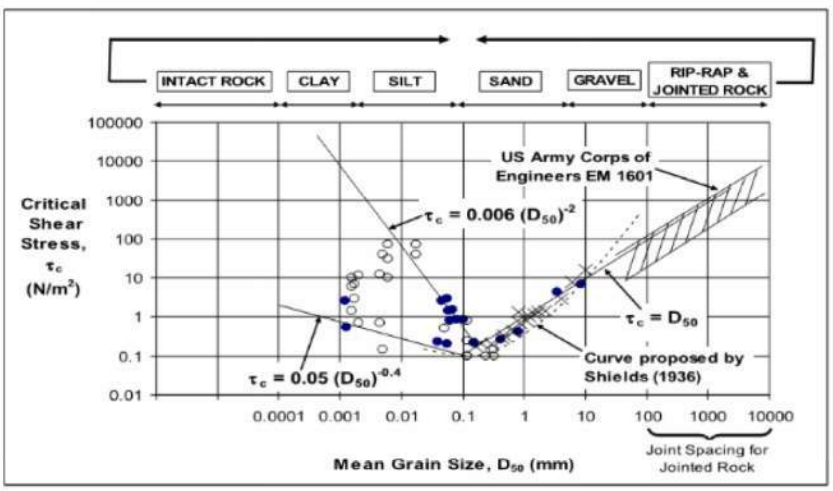


Figure 6.9. Critical shear stress versus particle size (Briaud et al. 2011).

D50 = 0.003 mm

HYDRAULIC DATA
Bridge Scour Variables

Storm Events being Analyzed:

$$Storm := \begin{bmatrix} 2 \\ 5 \\ 10 \\ 25 \\ 50 \\ 100 \\ 200 \\ 500 \end{bmatrix}$$

Max Velocity within 2 Flow Depths of Countermeasure Toe:
(flow depth is approximately the same as h)

$$V := \begin{bmatrix} 2.51 \\ 3.35 \\ 4.16 \\ 4.64 \\ 4.93 \\ 5.69 \\ 6.52 \\ 7.53 \end{bmatrix} \cdot \frac{ft}{s}$$

SMS Node 4274

Max Depth within 2 Flow Depths of Countermeasure Toe:

$$h := \begin{bmatrix} 13.70 \\ 15.49 \\ 15.66 \\ 17.27 \\ 18.42 \\ 18.54 \\ 20.02 \\ 20.15 \end{bmatrix} \cdot ft$$

SMS Node 4274

Max Unit Discharge within 2 Flow Depths of Countermeasure Toe:

$$q := \begin{array}{l} \text{for } i \in 1 .. \text{rows}(V) \\ \quad q_i \leftarrow V_i \cdot h_i \\ q \end{array} = \begin{bmatrix} 34.4 \\ 51.9 \\ 65.1 \\ 80.1 \\ 90.8 \\ 105.5 \\ 130.5 \\ 151.7 \end{bmatrix} \frac{ft^2}{s}$$

Shear Stress within 2 Flow Depths of Countermeasure Toe:

$$\tau_{initial} := \begin{array}{l} \text{for } i \in 1 .. \text{rows}(V) \\ \quad \tau_i \leftarrow \dfrac{\rho_w \cdot g \cdot \left(n_{man} \cdot V_i\right)^2}{K_u^{\,2} \cdot h_i^{\frac{1}{3}}} \\ \tau \end{array} = \begin{bmatrix} 0.036 \\ 0.062 \\ 0.095 \\ 0.114 \\ 0.126 \\ 0.167 \\ 0.214 \\ 0.285 \end{bmatrix} \frac{lbf}{ft^2}$$

Ref. 1, Eq. 6.7

D50 = 0.003 mm

TOTAL SCOUR
NCHRP 24-20 Abutment Scour Approach
Clear Water Scour:

Flow Depth Including Live-Bed Scour: $y_c := \left(\frac{\rho_w \cdot g}{\tau_c}\right)^{\frac{3}{7}} \cdot \left(\frac{n_{man} \cdot q}{K_u}\right)^{\frac{6}{7}} = \begin{bmatrix} 23.1 \\ 32.9 \\ 39.9 \\ 47.7 \\ 53.1 \\ 60.4 \\ 72.4 \\ 82.4 \end{bmatrix} ft$ Ref. 1, Eq. 8.7

Scour Amplification Factor for Clear-Water Conditions: $\alpha_{B.clear} := 1.13$ Ref. 1, Fig. 8.11 (As seen Below)
(flow within floodplain is much smaller than flow within bridge. Therefore q2f/qf approaches infinity.)

Maximum Flow Depth: $y_{max.clear} := \alpha_{B.clear} \cdot y_c = \begin{bmatrix} 26.1 \\ 37.1 \\ 45.1 \\ 53.9 \\ 60.0 \\ 68.2 \\ 81.9 \\ 93.1 \end{bmatrix} ft$ Ref. 1, Eq. 8.3

Abutment Scour Depth: $y_{s.NCHRP.clear} := y_{max.clear} - h$ Ref. 1, Eq. 8.4

$y_{s.NCHRP.clear} = \begin{bmatrix} 12.4 \\ 21.6 \\ 29.5 \\ 36.6 \\ 41.6 \\ 49.7 \\ 61.8 \\ 73.0 \end{bmatrix} ft$

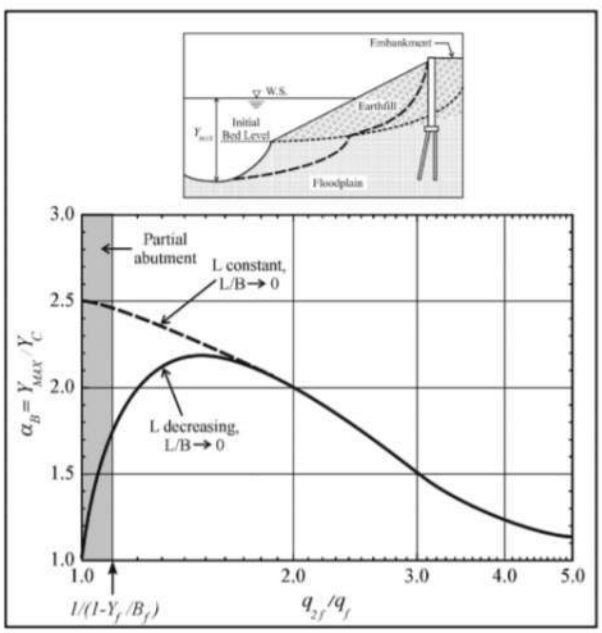


Figure 8.11. Scour amplification factor for spill-through abutments and clear-water conditions (NCHRP 2010b).

*****The existing bridge is not scour critical. Scour analysis is to be refined to account Rate of Scour with respect to life time of bridge.**

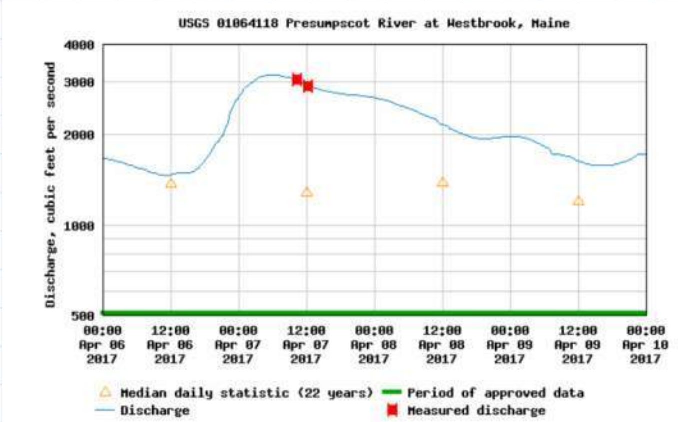
TOTAL SCOUR (CONT.)
Bridge Design Variables:

D50 = 0.003 mm

Bridge Design Life, yr: $Design_{life} := 75$

Duration of Hydrograph Peak Discharge: $Peak_{time} := 18\ hr$

Assumed based on limited USGS 01064118 Gage Data. Low flow peaks last approximately 12 hrs. Use 18 hours to account for transitional flows, ex. 280-yr flow



Cumulative Binomial Distribution: This is used to determine the probability and number of storms of interest the bridge experiences during design life. Iterate values untill cumulative probability, P = 0.9999 or less. Storm Event 50 through 500 cumulative probability may be less than 0.9999. Therefore, iterate values such that the next integer does not increase the probability.

Storm Event Occurrence within Design Life:

$$Total_{designlife} := \begin{bmatrix} 50 \\ 25 \\ 14 \\ 12 \\ 6 \\ 1 \\ 1 \\ 1 \end{bmatrix} \qquad Storm = \begin{bmatrix} 2.0 \\ 5.0 \\ 10.0 \\ 25.0 \\ 50.0 \\ 100.0 \\ 200.0 \\ 500.0 \end{bmatrix}$$

D50 = 0.003 mm

TOTAL SCOUR (CONT.)
Cumulative Binomial Distribution, Cont.:

$$P := \text{for } j \in 1..\text{rows}(Storm)\quad \begin{array}{l} m \leftarrow Total_{designlife_j} \\ p \leftarrow \dfrac{1}{Storm_j} \\ n \leftarrow Design_{life} \\ P_{old} \leftarrow 0 \\ \text{for } k \in 1..m \\ \quad \begin{array}{l} P_{new} \leftarrow \dfrac{n!}{k! \cdot (n-k)!} \cdot p^k \cdot (1-p)^{(n-k)} \\ P_{old} \leftarrow P_{new} + P_{old} \\ P_j \leftarrow P_{old} \\ P \end{array} \end{array} = \begin{bmatrix} 0.9988 \\ 0.9979 \\ 0.9928 \\ 0.9532 \\ 0.7795 \\ 0.3565 \\ 0.2588 \\ 0.1293 \end{bmatrix}$$

Ref. 4, Eqs. 4.4, 4.32

Total Storm Duration within Design Life:

$$Total_{storm\_peak} := Peak_{time} \cdot Total_{designlife} = \begin{bmatrix} 900.0 \\ 450.0 \\ 252.0 \\ 216.0 \\ 108.0 \\ 18.0 \\ 18.0 \\ 18.0 \end{bmatrix} hr$$

D50 = 0.003 mm

TOTAL SCOUR (CONT.)

Erodibility:

Assume that marine clay is on the border of High to Medium Erodibility Line.
Reference 1, Figure 6.11

$$Z := \begin{bmatrix} 1.308\ Pa & 0.1\ \dfrac{mm}{hr} \\ 40.128\ Pa & 1000\ \dfrac{mm}{hr} \end{bmatrix}$$

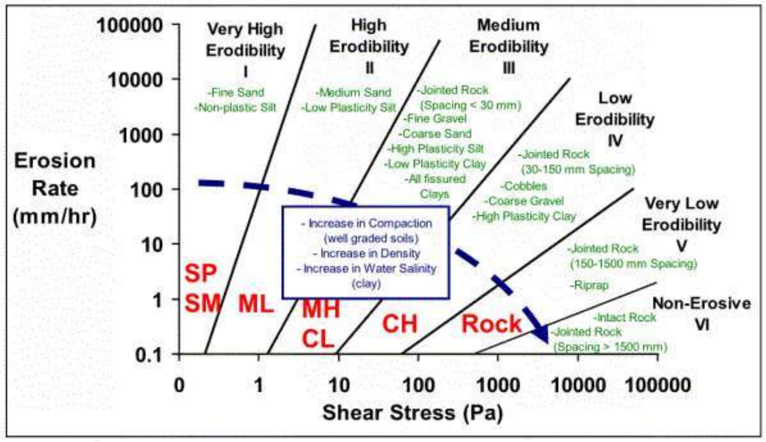


Figure 6.11. Generalized relationships for scour in cohesive materials (Briaud et al. 2011).

Calculate Erodibility, z_i

Use Logarithmic interpolation for Erosion Rates listed above to determine Erosion Rate for a specific Shear Stress.

Log-Log Slope:

$$m := \frac{\log\left(\dfrac{Z_{1,2}}{Z_{2,2}}\right)}{\log\left(\dfrac{Z_{1,1}}{Z_{2,1}}\right)} = 2.7$$

Log-Log Intercept:

$$b := \log\left(\frac{Z_{2,2} \cdot \dfrac{hr}{mm}}{\left(\dfrac{Z_{2,1}}{Pa}\right)^m}\right) = -1.3$$

Erosion Rate:

$$z_i := \left(\left(\frac{\tau_{initial}}{Pa}\right)^m \cdot 10^b\right) \cdot \frac{mm}{hr} = \begin{bmatrix} 0.001 \\ 0.003 \\ 0.009 \\ 0.015 \\ 0.020 \\ 0.043 \\ 0.084 \\ 0.181 \end{bmatrix} \frac{ft}{hr} \qquad z_i = \begin{bmatrix} 0.21 \\ 0.89 \\ 2.84 \\ 4.67 \\ 6.11 \\ 13.14 \\ 25.52 \\ 55.08 \end{bmatrix} \frac{mm}{hr} \quad \text{for} \quad \tau_{initial} = \begin{bmatrix} 1.73 \\ 2.95 \\ 4.53 \\ 5.46 \\ 6.03 \\ 8.02 \\ 10.26 \\ 13.66 \end{bmatrix} Pa$$

TOTAL SCOUR (CONT.)
Rate of Scour:

D50 = 0.003 mm

Calculate Scour Depth, yt
Ref. 1, Eqs. 6.8 & 6.9, Figure 6.12

$$
\begin{aligned}
y_t := &\text{ for } i \in 1..\text{rows}(\tau_{initial})\\
&\quad \text{if } i = 1 \wedge \tau_{initial_i} > \tau_c\\
&\quad\quad y_{t_i} \leftarrow \frac{Total_{storm\_peak_i}}{\dfrac{1}{z_{i_i}} + \dfrac{Total_{storm\_peak_i}}{y_{s.NCHRP.clear_i}}}\\
&\quad\quad y_t\\
&\quad \text{else if } i \geq 2 \wedge \tau_{initial_i} > \tau_{initial_{i-1}}\\
&\quad\quad T \leftarrow \frac{y_{s.NCHRP.clear_i} \cdot y_{t_{i-1}}}{z_{i_i} \cdot \left(y_{s.NCHRP.clear_i} - y_{t_{i-1}}\right)} + Total_{storm\_peak_i}\\
&\quad\quad y_{t_i} \leftarrow \frac{T}{\dfrac{1}{z_{i_i}} + \dfrac{T}{y_{s.NCHRP.clear_i}}}\\
&\quad\quad y_t\\
&\quad \text{else}\\
&\quad\quad y_{t_i} \leftarrow y_{t_{i-1}}
\end{aligned}
= \begin{bmatrix} 0.59291 \\ 1.77012 \\ 3.69678 \\ 6.17221 \\ 7.67565 \\ 8.22316 \\ 9.33280 \\ 11.71406 \end{bmatrix} ft
$$

SUMMARY

D50 = 0.003 mm

Clear Water Total Scour

Max Local Scour:

$$y_{s.NCHRP.clear} = \begin{bmatrix} 12.4 \\ 21.6 \\ 29.5 \\ 36.6 \\ 41.6 \\ 49.7 \\ 61.8 \\ 73.0 \end{bmatrix} ft \qquad Storm = \begin{bmatrix} 2.0 \\ 5.0 \\ 10.0 \\ 25.0 \\ 50.0 \\ 100.0 \\ 200.0 \\ 500.0 \end{bmatrix}$$

Local Scour within Design Life of Bridge:

$$y_t = \begin{bmatrix} 0.6 \\ 1.8 \\ 3.7 \\ 6.2 \\ 7.7 \\ 8.2 \\ 9.3 \\ 11.7 \end{bmatrix} ft \qquad Storm = \begin{bmatrix} 2.0 \\ 5.0 \\ 10.0 \\ 25.0 \\ 50.0 \\ 100.0 \\ 200.0 \\ 500.0 \end{bmatrix}$$

Scour Depth Elevation

Channel Thawleg Elevation: $El_{Thawleg} := 62.50\ ft$

Scour Elevation: $Scour_{el} := El_{Thawleg} - y_t$

$$Scour_{el} = \begin{bmatrix} 61.9 \\ 60.7 \\ 58.8 \\ 56.3 \\ 54.8 \\ 54.3 \\ 53.2 \\ 50.8 \end{bmatrix} ft \qquad Storm = \begin{bmatrix} 2.0 \\ 5.0 \\ 10.0 \\ 25.0 \\ 50.0 \\ 100.0 \\ 200.0 \\ 500.0 \end{bmatrix}$$

NOTES AND ASSUMPTIONS

$\boxed{D50 = 0.0003\ mm}$

References:

1. FHWA HEC 18, 5th Edition, Publication No. FHWA-HIF-12-003
2. MaineDOT Bridge Design Guide 2003 with updates to June 2018
3. Preliminary Geotechnical Letter Report prepared by Thielsch Engineering, Inc.
4. Water Resources Engineering, David A. Chin, Second Edition

- Scour is to be analyzed per FHWA Hydraulic Engineering Circular (HEC) 18.
- Proposed hydraulic data including flood velocity and elevations are taken from Proposed SRH-2D Model.
- The proposed bridge's hydraulic opening was designed for a 50 Year Flood Frequency. Per the MaineDOT Bridge Design Guide the 100 Year Flood is suggested for scour design and the 500 Year Flood is a check.
- Based on the lab results of the material from the borings taken by GZA, the bed material consists of marine clay. Therefore, clear water scour will occur. A sieve analysis was not performed on the bed material. It has been assumed that the HEC-18 Figure 6.9 will be used to estimate the grain size for the bed material. Figure 6.9 shows a transition between clay and silt at approximately 0.0003 mm. For this analysis, 0.0003 mm will be used to estimate critical shear stress.

GIVENS

$\boxed{D50 = 0.0003\ mm}$

Median Grain Size: $D_{50} := 0.0003\ mm$

Conversion to Customary Units: $K_u := 1.486$

Density of Water: $\rho_w := 1.94\ \frac{lbf \cdot s^2}{ft^4}$ (62.4 pcf)

Manning's n of Channel: $n_{man} := 0.022 \cdot \frac{s}{ft^{\frac{1}{3}}}$

Gravity: $g := 32.2\ \frac{ft}{s^2}$

Calculated Shear Stress: $\tau_c := 0.05 \left(\frac{N}{m^2}\right) \cdot \left(\frac{D_{50}}{mm}\right)^{-0.4} = 0.027\ \frac{lbf}{ft^2}$ Figure 6.9, Reference 1

(FHWA reccommends using the lower bound to compute shear stress)

Specific Gravity of Riprap: $S_s := 2.65$

Figure 6.9, Reference 1

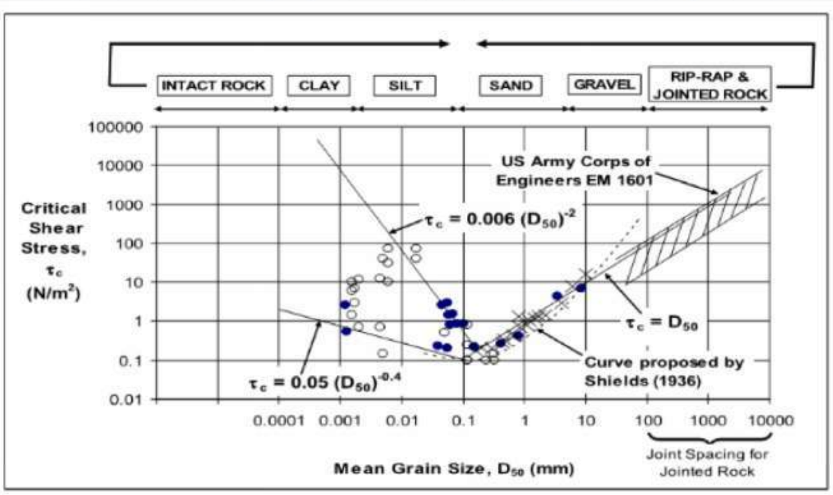


Figure 6.9. Critical shear stress versus particle size (Briaud et al. 2011).

D50 = 0.0003 mm

HYDRAULIC DATA
Bridge Scour Variables

Storm Events being Analyzed:

$$Storm := \begin{bmatrix} 2 \\ 5 \\ 10 \\ 25 \\ 50 \\ 100 \\ 200 \\ 500 \end{bmatrix}$$

Max Velocity within 2 Flow Depths of Countermeasure Toe:
(flow depth is approximately the same as h)

$$V := \begin{bmatrix} 2.51 \\ 3.35 \\ 4.16 \\ 4.64 \\ 4.93 \\ 5.69 \\ 6.52 \\ 7.53 \end{bmatrix} \cdot \frac{ft}{s}$$

SMS Node 4274

Max Depth within 2 Flow Depths of Countermeasure Toe:

$$h := \begin{bmatrix} 13.70 \\ 15.49 \\ 15.66 \\ 17.27 \\ 18.42 \\ 18.54 \\ 20.02 \\ 20.15 \end{bmatrix} \cdot ft$$

SMS Node 4274

Max Unit Discharge within 2 Flow Depths of Countermeasure Toe:

$$q := \begin{array}{l} \text{for } i \in 1 .. \text{rows}(V) \\ \quad q_i \leftarrow V_i \cdot h_i \\ q \end{array} = \begin{bmatrix} 34.4 \\ 51.9 \\ 65.1 \\ 80.1 \\ 90.8 \\ 105.5 \\ 130.5 \\ 151.7 \end{bmatrix} \frac{ft^2}{s}$$

Shear Stress within 2 Flow Depths of Countermeasure Toe:

$$\tau_{initial} := \begin{array}{l} \text{for } i \in 1 .. \text{rows}(V) \\ \quad \tau_i \leftarrow \dfrac{\rho_w \cdot g \cdot \left(n_{man} \cdot V_i\right)^2}{K_u^{\ 2} \cdot h_i^{\frac{1}{3}}} \\ \tau \end{array} = \begin{bmatrix} 0.036 \\ 0.062 \\ 0.095 \\ 0.114 \\ 0.126 \\ 0.167 \\ 0.214 \\ 0.285 \end{bmatrix} \frac{lbf}{ft^2}$$

Ref. 1, Eq. 6.7

D50 = 0.0003 mm

TOTAL SCOUR
NCHRP 24-20 Abutment Scour Approach
Clear Water Scour:

Flow Depth Including Live-Bed Scour: $y_c := \left(\frac{\rho_w \cdot g}{\tau_c}\right)^{\frac{3}{7}} \cdot \left(\frac{n_{man} \cdot q}{K_u}\right)^{\frac{6}{7}} = \begin{bmatrix} 15.6 \\ 22.1 \\ 26.9 \\ 32.1 \\ 35.8 \\ 40.7 \\ 48.8 \\ 55.5 \end{bmatrix} ft$ Ref. 1, Eq. 8.7

Scour Amplification Factor for Clear-Water Conditions: $\alpha_{B.clear} := 1.13$ Ref. 1, Fig. 8.11 (As seen Below)
(flow within floodplain is much smaller than flow within bridge. Therefore q2f/qf approaches infinity.)

Maximum Flow Depth: $y_{max.clear} := \alpha_{B.clear} \cdot y_c = \begin{bmatrix} 17.6 \\ 25.0 \\ 30.4 \\ 36.3 \\ 40.4 \\ 46.0 \\ 55.2 \\ 62.8 \end{bmatrix} ft$ Ref. 1, Eq. 8.3

Abutment Scour Depth: $y_{s.NCHRP.clear} := y_{max.clear} - h$ Ref. 1, Eq. 8.4

$y_{s.NCHRP.clear} = \begin{bmatrix} 3.9 \\ 9.5 \\ 14.7 \\ 19.0 \\ 22.0 \\ 27.4 \\ 35.1 \\ 42.6 \end{bmatrix} ft$

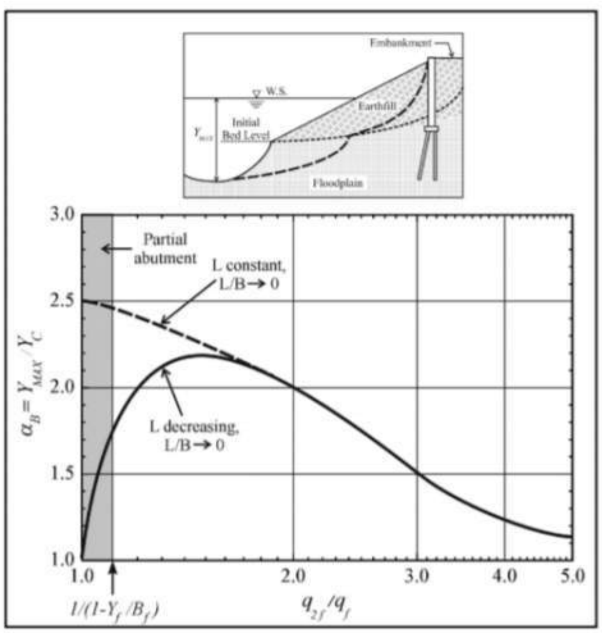


Figure 8.11. Scour amplification factor for spill-through abutments and clear-water conditions (NCHRP 2010b).

*****The existing bridge is not scour critical. Scour analysis is to be refined to account Rate of Scour with respect to life time of bridge.**

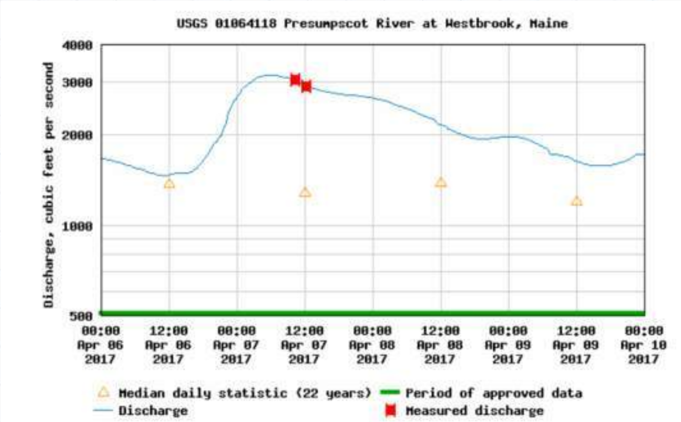
D50 = 0.0003 mm

TOTAL SCOUR (CONT.)
Bridge Design Variables:

Bridge Design Life, yr: $Design_{life} := 75$

Duration of Hydrograph Peak Discharge: $Peak_{time} := 18\ hr$

Assumed based on limited USGS 01064118 Gage Data. Low flow peaks last approximately 12 hrs. Use 18 hours to account for transitional flows, ex. 280-yr flow



Cumulative Binomial Distribution: This is used to determine the probability and number of storms of interest the bridge experiences during design life. Iterate values untill cumulative probability, P = 0.9999 or less. Storm Event 50 through 500 cumulative probability may be less than 0.9999. Therefore, iterate values such that the next integer does not increase the probability.

Storm Event Occurrence within Design Life:

$$Total_{designlife} := \begin{bmatrix} 50 \\ 25 \\ 14 \\ 12 \\ 6 \\ 1 \\ 1 \\ 1 \end{bmatrix} \qquad Storm = \begin{bmatrix} 2.0 \\ 5.0 \\ 10.0 \\ 25.0 \\ 50.0 \\ 100.0 \\ 200.0 \\ 500.0 \end{bmatrix}$$

D50 = 0.0003 mm

TOTAL SCOUR (CONT.)
Cumulative Binomial Distribution, Cont.:

$$P := \text{for } j \in 1..\text{rows}(Storm)$$
$$\quad m \leftarrow Total_{designlife_j}$$
$$\quad p \leftarrow \frac{1}{Storm_j}$$
$$\quad n \leftarrow Design_{life}$$
$$\quad P_{old} \leftarrow 0$$
$$\quad \text{for } k \in 1..m$$
$$\quad\quad P_{new} \leftarrow \frac{n!}{k! \cdot (n-k)!} \cdot p^k \cdot (1-p)^{(n-k)}$$
$$\quad\quad P_{old} \leftarrow P_{new} + P_{old}$$
$$\quad\quad P_j \leftarrow P_{old}$$
$$\quad\quad P$$
$$= \begin{bmatrix} 0.9988 \\ 0.9979 \\ 0.9928 \\ 0.9532 \\ 0.7795 \\ 0.3565 \\ 0.2588 \\ 0.1293 \end{bmatrix}$$

Ref. 4, Eqs. 4.4, 4.32

Total Storm Duration within Design Life:

$$Total_{storm\_peak} := Peak_{time} \cdot Total_{designlife} = \begin{bmatrix} 900.0 \\ 450.0 \\ 252.0 \\ 216.0 \\ 108.0 \\ 18.0 \\ 18.0 \\ 18.0 \end{bmatrix} hr$$

D50 = 0.0003 mm

TOTAL SCOUR (CONT.)

Erodibility:

Assume that marine clay is on the border of High to Medium Erodibility Line.
Reference 1, Figure 6.11

$$Z := \begin{bmatrix} 1.308\ Pa & 0.1\ \frac{mm}{hr} \\ 40.128\ Pa & 1000\ \frac{mm}{hr} \end{bmatrix}$$

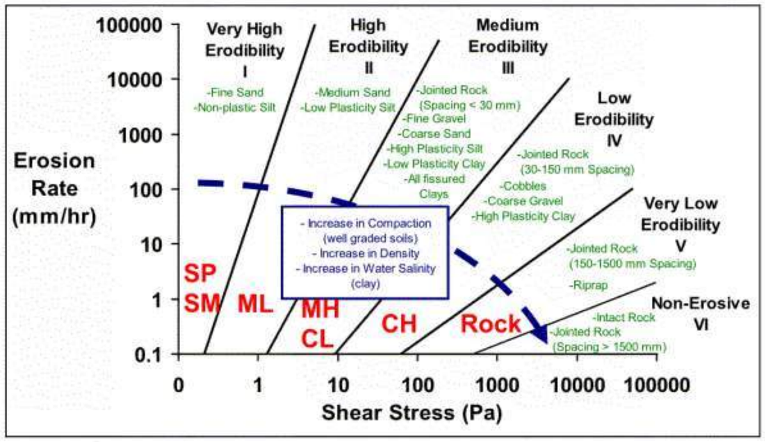


Figure 6.11. Generalized relationships for scour in cohesive materials (Briaud et al. 2011).

Calculate Erodibility, z_i

Use Logarithmic interpolation for Erosion Rates listed above to determine Erosion Rate for a specific Shear Stress.

Log-Log Slope:

$$m := \frac{\log\left(\frac{Z_{1,2}}{Z_{2,2}}\right)}{\log\left(\frac{Z_{1,1}}{Z_{2,1}}\right)} = 2.7$$

Log-Log Intercept:

$$b := \log\left(\frac{Z_{2,2} \cdot \frac{hr}{mm}}{\left(\frac{Z_{2,1}}{Pa}\right)^m}\right) = -1.3$$

Erosion Rate:

$$z_i := \left(\left(\frac{\tau_{initial}}{Pa}\right)^m \cdot 10^b\right) \cdot \frac{mm}{hr} = \begin{bmatrix} 0.001 \\ 0.003 \\ 0.009 \\ 0.015 \\ 0.020 \\ 0.043 \\ 0.084 \\ 0.181 \end{bmatrix} \frac{ft}{hr} \qquad z_i = \begin{bmatrix} 0.21 \\ 0.89 \\ 2.84 \\ 4.67 \\ 6.11 \\ 13.14 \\ 25.52 \\ 55.08 \end{bmatrix} \frac{mm}{hr} \quad \text{for} \quad \tau_{initial} = \begin{bmatrix} 1.73 \\ 2.95 \\ 4.53 \\ 5.46 \\ 6.03 \\ 8.02 \\ 10.26 \\ 13.66 \end{bmatrix} Pa$$

D50 = 0.0003 mm

TOTAL SCOUR (CONT.)
Rate of Scour:

Calculate Scour Depth, yt
Ref. 1, Eqs. 6.8 & 6.9, Figure 6.12

$y_t :=$ for $i \in 1..\text{rows}(\tau_{initial})$
  if $i = 1 \wedge \tau_{initial_i} > \tau_c$
    $y_{t_i} \leftarrow \dfrac{Total_{storm\_peak_i}}{\dfrac{1}{z_{i_i}} + \dfrac{Total_{storm\_peak_i}}{y_{s.NCHRP.clear_i}}}$
    y_t
  else if $i \geq 2 \wedge \tau_{initial_i} > \tau_{initial_{i-1}}$
    $T \leftarrow \dfrac{y_{s.NCHRP.clear_i} \cdot y_{t_{i-1}}}{z_{i_i} \cdot \left(y_{s.NCHRP.clear_i} - y_{t_{i-1}}\right)} + Total_{storm\_peak_i}$
    $y_{t_i} \leftarrow \dfrac{T}{\dfrac{1}{z_{i_i}} + \dfrac{T}{y_{s.NCHRP.clear_i}}}$
    y_t
  else
    $y_{t_i} \leftarrow y_{t_{i-1}}$

$$= \begin{bmatrix} 0.53662 \\ 1.57495 \\ 3.21236 \\ 5.21168 \\ 6.38457 \\ 6.83160 \\ 7.77713 \\ 9.82292 \end{bmatrix} ft$$

SUMMARY

D50 = 0.0003 mm

Clear Water Total Scour

Max Local Scour:

$$y_{s.NCHRP.clear} = \begin{bmatrix} 3.9 \\ 9.5 \\ 14.7 \\ 19.0 \\ 22.0 \\ 27.4 \\ 35.1 \\ 42.6 \end{bmatrix} ft \qquad Storm = \begin{bmatrix} 2.0 \\ 5.0 \\ 10.0 \\ 25.0 \\ 50.0 \\ 100.0 \\ 200.0 \\ 500.0 \end{bmatrix}$$

Local Scour within Design Life of Bridge:

$$y_t = \begin{bmatrix} 0.5 \\ 1.6 \\ 3.2 \\ 5.2 \\ 6.4 \\ 6.8 \\ 7.8 \\ 9.8 \end{bmatrix} ft \qquad Storm = \begin{bmatrix} 2.0 \\ 5.0 \\ 10.0 \\ 25.0 \\ 50.0 \\ 100.0 \\ 200.0 \\ 500.0 \end{bmatrix}$$

Scour Depth Elevation

Channel Thawleg Elevation: $El_{Thawleg} := 62.50\ ft$

Scour Elevation: $Scour_{el} := El_{Thawleg} - y_t$

$$Scour_{el} = \begin{bmatrix} 62.0 \\ 60.9 \\ 59.3 \\ 57.3 \\ 56.1 \\ 55.7 \\ 54.7 \\ 52.7 \end{bmatrix} ft \qquad Storm = \begin{bmatrix} 2.0 \\ 5.0 \\ 10.0 \\ 25.0 \\ 50.0 \\ 100.0 \\ 200.0 \\ 500.0 \end{bmatrix}$$